



Geotechnical Investigation

Midtown East Parking Structure
Winsor Street and Weller Lane
Milpitas, California

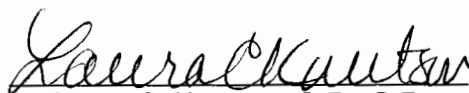
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TABLE OF CONTENTS

1.0	INTRODUCTION.....	1
1.1	Project Background.....	1
1.2	Project Description.....	1
1.3	Scope of Services	2
2.0	SITE CONDITIONS	2
2.1	Exploration Program	2
2.2	Surface	3
2.3	Subsurface	3
2.4	Ground Water	3
2.5	Site Infiltration.....	4
3.0	GEOLOGIC HAZARDS	4
3.1	Fault Rupture Hazard	4
	Table 1. Approximate Distance to Seismic Sources.....	4
3.1.1	Maximum Estimated Ground Shaking.....	5
3.1.2	Future Earthquake Probabilities.....	5
3.2	Liquefaction	5
3.2.1	Analyses and Results	5
	Table 2. Results of Liquefaction Analyses	7
3.2.2	Potential for Ground Rupture/Sand Boils	8
3.3	Seismically-Induced Settlements	8
3.4	Lateral Spreading	8
3.5	Flooding	9
4.0	CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS	9
4.1	Conclusions	9
4.1.1	Liquefaction-Induced Settlements	9
4.1.2	Moderately Compressible Soils.....	9
4.1.3	Former UST Excavations	10
4.1.4	Shallow Ground Water	10
4.1.5	Moderate to High Expansion Potential of Surficial Soils.....	10
4.2	Plans, Specifications, and Construction Review.....	11
5.0	EARTHWORK.....	11

5.1	Clearing and Site Preparation	11
5.2	Removal of Undocumented Fill in Former UST Excavations	11
5.3	Abandoned Utilities	12
5.4	Subgrade Preparation	12
5.5	Material for Fill	12
5.6	Compaction	13
5.7	Unstable Soil Conditions	13
5.8	Lime Treatment	14
5.9	Trench Backfill	14
5.10	Temporary Dewatering	15
5.11	Temporary Slopes and Trench Excavations	15
5.12	Surface Drainage	15
5.13	Landscaping Considerations	16
5.14	Construction Observation	16
6.0	1997 UBC/2001 CBC SITE SEISMIC COEFFICIENTS	16
	Table 3. 1997 UBC/2001 CBC Site Categorization and Seismic Coefficients ..	17
7.0	DEEP FOUNDATIONS	17
7.1	Downdrag on Piles	17
7.2	Driven Pre-Cast Concrete Piles	18
	7.2.1 Lateral Loads on Driven Piles	19
	Table 4. Estimated Lateral Pile Response – 14" Square Piles	19
	7.2.2 WEAP Analysis	20
	7.2.3 Indicator Piles	20
	7.2.4 PDA Monitoring	20
	7.2.5 Production Pile Installation	21
7.3	Displacement Augercast Piles	21
	7.3.1 Vertical Loads	22
	7.3.2 Lateral Loads on Augercast Piles	22
	Table 5. Estimated Lateral Pile Response – 16" Round Piles	23
	7.3.3 Pile Load Tests	24
7.4	Interior Slabs-On-Grade	24
7.5	Building Pad Moisture Conditioning	25
7.6	Moisture Protection Considerations	25

8.0	AT-GRADE SITE RETAINING WALLS	26
8.1	Lateral Earth Pressures.....	26
8.2	Drainage	26
8.3	Backfill.....	27
8.4	Foundation	27
8.5	Lateral Loads	28
9.0	AT-GRADE PAVEMENTS	28
9.1	Asphalt Concrete	28
	Table 6. Recommended Asphalt Concrete Pavement Design Alternatives.....	28
9.2	Exterior Portland Cement Concrete	29
	Table 7. Recommended Minimum PCC Pavement Thickness	29
9.3	Concrete Pavers	29
9.4	Pavement Cutoff.....	30
9.5	Asphalt Concrete, Aggregate Base and Subgrade.....	30
9.6	Exterior Concrete Flatwork.....	30
10.0	LIMITATIONS	31
11.0	REFERENCES.....	32

FIGURE 1 — VICINITY MAP

FIGURE 2 — SITE PLAN

FIGURE 3 — REGIONAL FAULT MAP

FIGURE 4 — VERTICAL PILE CAPACITY CHART

APPENDIX A — FIELD INVESTIGATION

APPENDIX B — LABORATORY PROGRAM

APPENDIX C — PREVIOUS FIELD INVESTIGATION DATA BY OTHERS

**GEOTECHNICAL INVESTIGATION
MIDTOWN EAST PARKING STRUCTURE
WINSOR STREET AND WELLER LANE
MILPITAS, CALIFORNIA**

1.0 INTRODUCTION

In this report, we present the results of our geotechnical investigation for the Midtown East Parking Structure to be located in Milpitas, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design and construction of the proposed development.

For our use, we received the following:

- A conceptual site plan in pdf format from Chong Partners Architecture, undated.
- A Site Plan titled "City of Milpitas East Parking Garage," prepared by Chong Partners Architecture, dated January 19, 2006.
- A geotechnical report prepared for the combined public library and parking structure site prepared by Treadwell & Rollo, Inc., titled "Geotechnical Investigation, Milpitas Library, Milpitas, California," dated September 14, 2004.

1.1 Project Background

As you know, we previously performed an environmental study for an adjacent site immediately north of the project site for the City of Milpitas. The results of the findings are presented in our October 1, 2004 report titled "Soil Quality Evaluation and Geophysical Survey, Milpitas Library: Apton Parcel, Milpitas, California." As part of this environmental study, we were provided with environmental reports that were previously prepared for the Midtown East Parking Structure.

Based on our review of the environmental reports, we understand that three former underground storage tank (UST) excavations are located within the proposed Midtown East Parking Structure area. The depths of the UST excavations were reported to be on the order of 9½ to 10 feet, and they were also reported to be backfilled with imported fill; however, no records of compaction testing were included with the documents that were available for our review at the time this report was prepared. Ground water levels were recorded at depths between 5 and 8.8 feet at the site. The approximate locations of the UST excavations are shown on the Site Plan, Figure 2.

1.2 Project Description

As presently planned, the project consists of demolishing the existing structures and developing the site with a 3-story, 4-level, at-grade, concrete-frame parking structure and possibly with retail/commercial space located at ground level. We understand that the City will evaluate whether or not to include the retail/commercial space during

the design. Associated underground utilities, pavements and landscaping are also planned as part of the site development. The approximate layout of the proposed building footprint is shown on the Site Plan, Figure 2.

Structural loads provided by the project structural engineer, Mr. Ken Napier of Walker Parking Consultants, indicate that exterior, interior and maximum dead plus live column loads for the 3-story parking structure are on the order of 210 to 920 kips. We anticipate that only minor grading will be required.

1.3 Scope of Services

Our scope of services was presented in detail in our agreement with you dated March 2, 2005. To accomplish this work, we provided the following services:

- Review of geotechnical and environmental reports previously prepared for site and adjacent sites.
- Exploration of subsurface conditions by drilling two borings and retrieving soil samples for visual observation and laboratory testing. Eight Cone Penetration Tests (CPTs) were also advanced.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples. Correlation of CPT interpretations with visual classification and laboratory testing on samples collected from our borings.
- Engineering analysis to evaluate site earthwork, building foundations, slabs-on-grade, retaining walls and pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

Environmental services were not included as part of this study.

2.0 SITE CONDITIONS

2.1 Exploration Program

Subsurface explorations were performed on May 11, 16 and 20, 2005, and January 20, 2006 using conventional, truck-mounted CPT, mud-rotary, and hollow-stem auger drilling equipment to investigate, sample, and log subsurface soils. Two exploratory borings were drilled to depths of 43½ and 100 feet. In addition, eight CPTs were advanced to depths ranging from 30 to 100 feet. Our borings and CPTs were permitted and backfilled in accordance with Santa Clara Valley Water District guidelines. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. Logs of our borings and CPTs, and details regarding our field investigation are included in Appendix A; our laboratory tests are discussed in Appendix B. Previous field investigation and laboratory testing data for the site by others are attached in Appendix C.

2.2 Surface

We also performed a brief surface reconnaissance during our site exploration. The site is located in a commercial/industrial area and is located on both sides of Winsor Street in Milpitas, California. At the time of our initial field exploration, the eastern half of the site was occupied by two single-story industrial buildings and Winsor Street pavements. The western half of the site consists of three parcels. The northernmost parcel is currently the City's maintenance yard and was occupied by several City vehicles, shopping carts, a truck container, and a shed; the middle parcel was occupied by two sheds, two vehicles, and a truck container; and the southernmost parcel was occupied by a boat and two vehicles. During our recent field investigation on the parcels that the City of Milpitas was not own and could not provide us access previously, we noticed that the buildings and structures at the site were under demolition. Topographic information was not available at the time this report was prepared. Based on our observations, the site appears to be relatively level.

2.3 Subsurface

Based on our explorations, the alluvial soils encountered generally consisted of predominantly medium stiff to hard clays to a maximum depth explored of 100 feet within the northernmost portion of the site. The remainder of the site, however, is underlain by medium stiff to hard clays with interbedded sand layers with variable quantities of silt and clay fines to a maximum depth explored of 100 feet (the hatched area on the Site Plan, Figure 2). The sand layers encountered varied in density from medium dense to very dense throughout the soil column.

A Plasticity Index (PI) test was performed on a near-surface soil sample. The test result exhibited a PI of 41, indicating the near-surface soils have high plasticity and expansion potential.

In general, the subsurface conditions encountered during our investigation correlate well with the explorations previously performed at the site by Treadwell & Rollo (T&R). Previous PI tests performed by T&R on the near-surface soil samples at the site exhibited PI's in the range of 34 to 44.

2.4 Ground Water

Ground water was measured at a depth of about 10½ feet in Boring EB-2 during drilling. We could not measure the ground water level in the mud-rotary boring due to establishment of the circulation drilling method at shallow depths. Pore pressure measurements taken in our CPTs indicated the depth to the ground water level is at depths between about 5½ to 7½ feet below the existing ground surface. Please note the ground water depth measurements were taken at the time of our exploration and may not reflect a stabilized level. All borings and CPTs were immediately backfilled after the exploration.

Environmental reports previously prepared for the site by others recorded ground water levels between depths of about 5 to 8.8 feet below the existing ground surface. The groundwater map published by the California Geological Survey (CGS) indicates historical high ground water level at the site to be less than 5 feet below the ground surface. Based on this information, a design ground water level of 4 feet was used for our liquefaction analyses. Fluctuations in the level of the ground water may occur due

to variations in rainfall, underground drainage patterns, and other factors not evident at the time we performed our explorations.

2.5 Site Infiltration

Our explorations indicate that the site is blanketed by about 3 to 9 feet of high plasticity clays. Generally, the higher the plasticity, the lower the permeability and hydraulic conductivity. Therefore, we judge the site infiltration rate will be very low for any proposed site detention/retention facilities. The Regional Water Quality Control Board (RWQCB) requires that a minimum of 10 feet be maintained between the seasonal high ground water level and the bottom of any infiltration facility. As discussed above, ground water was encountered at a depth as shallow as 5½ feet below the existing site grade. Therefore, pre-treatment of pavement runoff would likely be required.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. Table 1 lists the faults within 25 kilometers of the site:

Table 1. Approximate Distance to Seismic Sources

Fault	Seismic Source Type	Distance (miles)	Distance (kilometers)
*Hayward (Southeast Extension)	B	1.9	3.1
**Hayward (Total Length)	A	4.4	7.1
Calaveras	B	5.5	8.8
Monte Vista - Shannon	B	12.3	19.6
San Andreas	A	15.8	25.3

*Nearest Type B fault

**Nearest Type A fault

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone) nor is it located in a Santa Clara County Fault Rupture Hazard Zone (SCC, 2002). As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.1.1 Maximum Estimated Ground Shaking

The Probabilistic Seismic Hazard Analysis (PSHA) performed by the CGS estimates a pseudo-peak horizontal ground acceleration of 0.55g for the site with a 10 percent probability of exceedance in 50 years. Pseudo-peak ground accelerations have been normalized to a 7.5Mw seismic event, weighted to account for regional seismic activity and fault distances.

3.1.2 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2003), referred to as WG02, estimates there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2002 and 2031. This result is an important outcome of WG02's work because any major earthquake can cause damage throughout the region.

The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco located more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.2 Liquefaction

The site is located within an area zoned by the State of California as having potential for seismically induced liquefaction hazards (CGS, 2001 – Milpitas Quadrangle) and in a Santa Clara County Liquefaction Hazard Zone (SCC, 2002) mapped liquefaction zone. During cyclic ground shaking such as earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix resulting in liquefaction. Liquefied soil may lose shear strength and result in large shear deformations and/or flow failure under moderate to high shear stresses, such as beneath foundations or sloping ground (Youd et al., 2001). Liquefied soil can also settle (compact) as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

Soils most susceptible to liquefaction are loose to moderately dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

3.2.1 Analyses and Results

As noted in the subsurface description above, several cohesionless sand and silt layers were encountered below the recommended design ground water depth of 4 feet. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed structure.

Our liquefaction analyses followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in CDMG Special Publication 117 (CDMG, 1997). The NCEER methods for SPT and CPT analyses update simplified procedures presented by Seed and Idriss (1971).

In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. CRR calculations are based on SPT blow counts and CPT tip resistance. To account for effective overburden stresses and soil behavior, we corrected the field measured SPT blow counts for overburden, stress reduction versus depth, fine-grained soil content, hammer energy ratio, boring diameter, rod length and sampling method (SPT sampler without liners). Our CPT tip pressures were corrected for overburden and fines content. The CPT method utilizes the soil behavior type index (I_c) and the exponential factor "n" applied to the Normalized Cone Resistance "Q" to evaluate how plastic the soil behaves.

The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65 \left(\frac{a_{max}}{g} \right) \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) r_d$$

where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. We used a pseudo-peak horizontal acceleration of 0.55g, corresponding to a 10 percent chance of exceedance in 50 years for our analyses.

Soils that have significant amounts of plastic fines (greater than 25 percent), an I_c greater than 2.6, corrected SPT blow counts greater than 30 blows per foot, or corrected CPT tip resistances greater than 160 are considered either too plastic or too dense to liquefy. Such soil layers have been screened out of our analyses and are not presented below.

The factor of safety (FS) against liquefaction can be expressed as the ratio of the CRR to CSR. If the FS for a soil layer is less than 1.0, the soil layer may liquefy during a moderate to large seismic event.

$$FS = \frac{CRR}{CSR}$$

A summary of our analyses is presented in the table below. An analysis was not performed on the SPT data collected in the hollow stem boring since blow counts in hollow stem borings may be unreliable in sands below the ground water table. In addition, the hollow-stem boring was performed alongside of CPT-1 primarily to obtain soil samples for laboratory testing and classification confirmation.

Table 2. Results of Liquefaction Analyses

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	I _c	*q _{C1N-CS} (tsf)	Factor of Safety	Estimated Total Settlement (in.)	Estimated Differential Settlement (in.)
CPT-1	16.1	1.5	2.1	91	0.3	0.5	0.2
CPT-1	18	1	2.2	66	0.2	0.4	0.2
CPT-1	20.5	6	2.2	84	0.2	2.0	1.0
CPT-1	27	0.8	2.2	75	0.2	0.3	0.2
CPT-1	28.2	1	1.9	132	0.5	0.2	0.1
CPT-1	30	1	1.7	144	0.6	0.2	0.1
CPT-1	38.7	4.3	2.2	142	0.6	0.9	0.4
CPT-1	46.9	0.3	2.0	155	0.8	<0.1	--
Total =						4.5	2.2
CPT-2	15.2	0.7	2.3	70	0.2	0.2	0.1
CPT-2	16.1	11.3	1.9	105	0.3	3.2	1.6
CPT-2	29.5	0.3	1.3	158	0.7	<0.1	--
CPT-2	30.7	0.5	2.0	147	0.6	0.1	0.1
CPT-2	37.4	0.5	2.1	144	0.6	0.1	0.1
CPT-2	42.6	0.3	2.4	145	0.7	<0.1	--
CPT-2	48.2	0.3	2.1	126	0.5	<0.1	--
Total =						3.8	1.9
CPT-5	14	1.5	2.2	93	0.3	0.5	0.2
Total =						0.5	0.2
CPT-6	19	6.5	1.7	108	0.4	1.8	0.9
CPT-6	45	3.5	1.7	139	0.6	0.6	0.3
Total =						2.4	1.2
CPT-7	4	12	2.2	92	0.3	3.8	1.9
CPT-7	17.2	9.7	2.0	102	0.3	2.8	1.4
CPT-7	45.6	1.2	1.9	87	0.3	0.4	0.2
Total =						7.0	3.5
CPT-8	13	2.6	2.2	81	0.2	0.9	0.5
CPT-8	19.5	1.2	2.3	89	0.3	0.4	0.2
CPT-8	21.8	2.1	2.1	133	0.5	0.5	0.2
CPT-8	43.8	1.2	2.2	137	0.6	0.2	0.1
Total =						2.0	1.0

*Notes: CPT adjusted for overburden pressure and fines content.

Our analyses indicate that several silt and sand layers theoretically can liquefy, resulting in up to about 7 inches of total settlement. Estimates of volumetric change and settlement were calculated by the Ishihara and Yoshimine (1990) method. As discussed in the SCEC report, differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement.

Borings and CPTs previously performed by Treadwell & Rollo (T&R) for the site also encountered similar materials as our explorations to a maximum depth of 100 feet. T&R reported that granular soil was present at a depth of about 13½ to 21 feet below the existing ground surface and ranged from about 3½ to 20½ feet thick. T&R also reported that the upper part of the granular soil layer is loose to medium dense, and the lower portion is medium dense to dense. T&R concluded that an approximately 2- to 8½-foot-thick layer of loose to medium dense granular soil below the upper clay layers was susceptible to liquefaction during a moderate to large earthquake.

3.2.2 Potential for Ground Rupture/Sand Boils

The methods of analysis used to estimate total settlement do not take into account the possibility of surface ground rupture. In order for liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must exert a large enough force to break through the surface layer. For the southern 80 percent of the site (hatched area on Figure 2) where liquefaction is anticipated, there is approximately 4 to 19 feet of non-liquefiable material overlying about 15 feet of potentially liquefiable soils. Based on work by Youd and Garriss (1995), theoretically, there is not a thick enough non-liquefiable material cap to prevent ground rupture at the site; however, the sands from 4 to about 22 feet below grade contain over 30 percent passing the #200 sieve and while they may undergo liquefaction, they may not be able to release water fast enough to allow boils to develop. Therefore, we anticipate that the potential for ground rupture will be low at the site. Detailed recommendations regarding liquefaction mitigation are presented in the "Foundations" section of this report.

3.3 Seismically-Induced Settlements

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform settlement of soil strata, resulting in movement of the near-surface soils. As discussed in the liquefaction, there is a possibility of differential static and seismic movement between the southern and northern portions of the proposed parking garage. Provided the recommendations in the "Foundations" section of this report are followed, we judge the probability of significant differential compaction affecting the parking structure to be low.

3.4 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

The eastern side of the site is located about 100 feet west of the Berryessa Creek. Generally, the creek is about 5 to 10 feet below site grades and is not concrete-lined. In our opinion, the potential for lateral spreading is moderate along this portion of the creek; however, as the proposed site improvements will be at least about 100 feet away from the creek, we judge that the probability of lateral spreading affecting the site during a seismic event is low.

3.5 Flooding

As shown on the July 4, 1988 Federal Emergency Management Agency (FEMA) "Flood Insurance Rate Map" (FIRM), this site is within Zone AO, described as "Area of 100-year shallow flooding where depths are between one (1) and three (3) feet; average depths of inundation are shown, but no flood hazard factors are determined."

4.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

4.1 Conclusions

From a geotechnical engineering viewpoint the proposed development may be constructed as planned, in our opinion, provided the design is performed in accordance with the recommendations presented in this report.

The primary geotechnical and geologic concerns at the site are as follows:

- Liquefaction-induced settlements
- Moderately compressible soils
- Former UST excavations
- Shallow ground water
- Expansive surficial soils

For this report, we have prepared a brief description of the issues and presented typical approaches to manage potential concerns associated with the long-term performance of the development.

4.1.1 Liquefaction-Induced Settlements

Liquefaction-induced settlement is the primary concern for this site. As discussed in the "Liquefaction" section of this report, several silt and sand layers theoretically can liquefy, resulting in about 3¾ to 7 inches of total settlement across the southern portion of the parking structure (hatched area on Figure 2). To reduce damage to the planned parking structure, the building should be supported on deep foundations and that considerations should be given to designing the garage slab as a structural slab. At a minimum, the elevator pit and equipment room slabs should be structurally supported. Deep foundations will need to be designed to accommodate down-drag forces or include ground improvement around the structural piles to mitigate the liquefaction potential. Detailed recommendations addressing these concerns are presented in the following sections of this report.

4.1.2 Moderately Compressible Soils

The site is underlain by moderately compressible clays located within shallow foundation influence zones. We utilized the laboratory consolidation tests performed

by T&R as well as performing one additional test to evaluate the compressibility and strength of the clays; the results are attached in Appendices B and C. We performed settlement analyses using the structural column loads provided by the project structural engineer and an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus live loads. Our analyses indicate that static settlement will be on the order of 1 to 2 inches with differential movement estimated to be about 1-inch between columns. As discussed in the section above, liquefaction-induced settlements are also anticipated.

Based on our conversations with the project structural engineer, supporting the structure on deep foundations is planned due to the magnitude of the estimated settlements. Detailed recommendations are provided in the "Foundations" section of this report.

4.1.3 Former UST Excavations

Based on our review of previous environmental reports, three former USTs were excavated and backfilled. Figure 2 shows the approximate location of these excavation areas, which extended to depths of about 9½ to 10 feet below the existing ground surface. Since no records of compaction observation and testing were available for our review, we recommend that these former UST excavations be removed and replaced with compacted fill to reduce damage to the planned slabs-on-grade or pavements. However, if the owner is willing to accept some risk of slab and/or pavement cracking, and future maintenance, these former UST excavations may be sub-excavated to at least 3 feet below the slab-on-grade or pavement subgrade and replaced with engineered fill; fill within pile cap areas do not need to be removed. Detailed recommendations are discussed further in following sections of this report.

4.1.4 Shallow Ground Water

As discussed in the "Ground Water" section above, historically high ground water is reportedly as shallow as less than 5 feet below the existing site grades. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable subgrade soils, difficulty achieving compaction, and difficult underground utility installation. Therefore, the contractor should be aware that excavations extending near or below ground water might need to be stabilized and/or dewatered to facilitate placement and compaction of structures and backfill.

4.1.5 Moderate to High Expansion Potential of Surficial Soils

As discussed in the "Subsurface" section above, highly expansive surficial soils were encountered at the site. Expansive soils can undergo significant volume change with changes in the moisture content. Since the parking structure will be supported on deep foundations, surficial expansive soils should not impact the foundation performance. However, the planned garage slab-on-grade, at-grade sidewalks and flatwork should have sufficient reinforcement and be supported on a layer of non-expansive fill to reduce the potential for damage due to soil heaving and shrinkage. Detailed recommendations are presented in the following sections of this report.

4.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings and CPTs may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and test the geotechnical aspects of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation, and if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

5.0 EARTHWORK

5.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including any existing building foundations, slabs, pavements, and debris. Abandonment of existing buried utilities and removal of fills are discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of buried structures be carried out under our observation and that backfill be tested during placement.

5.2 Removal of Undocumented Fill in Former UST Excavations

As previously discussed, three former UST excavations are located in the proposed parking structure footprint. If the owner is willing to accept some risk of slab and/or pavement cracking and future maintenance, these former UST excavations may be excavated to a depth of 3 feet below the slab-on-grade and/or pavement subgrade and replaced with compacted fill. If this risk is not acceptable to the owner, all undocumented fill should be removed down to the native soil and replaced with engineered fill. Please note that the ground water level could be encountered at a depth less than 5 feet below the ground surface (CGS, 2001). Therefore, excavation extends below the ground water level will require dewatering. In addition, the excavated fill below the ground water table will likely be saturated and may not be suitable for use as engineered fill. If desired to re-use the excavated soils, drying of the soils or lime-treatment would be required. Recommendations for construction dewatering are presented in the following section of this report.

Side slopes of fill excavations should be sloped at inclinations no steeper than 3:1 (horizontal to vertical) to reduce the potential for distress to adjacent sidewalks and pavements. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section.

5.3 Abandoned Utilities

Abandoned utilities within the proposed parking structure area should be removed in their entirety. Existing underground utilities outside the proposed parking structure area should be removed or abandoned in-place by grouting or plugging the ends with concrete. The decision to abandon in-place versus removal should be based on the level of risk associated with the particular utility line.

It should be noted that fills associated with underground utilities abandoned in-place may have an increased potential for settlement, and partially grouted or plugged pipelines will have a potential risk of collapse that may result in ground settlement, soil piping, and leakage of pipeline constituents. The potential risks are relatively low for small diameter pipes (4 inches or less) and increasingly higher with increasing diameter.

5.4 Subgrade Preparation

After the site has been properly cleared, stripped, and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, foundations, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section.

The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment. If the relative compaction of the subgrade is less than recommended or the subgrade has significant yielding, then it should be reworked or over-excavated and rebuilt until the subgrade conforms to our recommendations.

Based on our laboratory test results, the native soils are about 10 to 17 percent over the estimated laboratory optimum moisture content. Earthwork contractor should anticipate that these soils would require drying (aeration) prior to use as engineered fill or subgrade preparation. Consideration should be given to the use of light-weight grading equipment and minimizing the concentration of rubber-tired equipment patterns during construction. The use of heavy equipment will tend to de-stabilize clays with high in-situ moisture contents. Contractors should also consider using only sheepsfoot compaction equipment, as vibratory equipment will also tend to de-stabilize clays with high in-situ moisture contents.

5.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Imported and non-expansive fill materials should be inorganic and should have a Plasticity Index of 15 or less. Imported fill should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Samples of proposed import fill should be submitted to us at least 10 days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials.

5.6 Compaction

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content near the laboratory optimum, except for the native expansive clays. The native expansive clays should be compacted to between 87 and 92 percent relative compaction at a moisture content at least 3 percent over optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and non-yielding under the weight of construction equipment.

Since the native soils have relatively high moisture contents, earthwork contractors should anticipate that these soils may require drying (aeration) prior to use as engineered fill or subgrade preparation even during summer months. Consideration should be given to the use of light-weight grading equipment and minimizing the concentration of rubber-tired equipment patterns during construction. The use of heavy and/or vibratory compaction equipment will tend to de-stabilize clays with high in-situ moisture contents.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition), except for the native clays, which should be compacted as noted above. Aggregate base and all import soils should be compacted at a moisture content slightly above the laboratory optimum.

5.7 Unstable Soil Conditions

It should be understood that earthwork such as fill placement and trench backfill may be very difficult during wet weather, especially for fill materials with a significant amount of clay. If the percent water in the fill increases significantly above the optimum moisture content, the soils will become soft, yielding, and difficult to compact. Therefore, we recommend that earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.

As discussed in the "Compaction" section, the in-situ moistures are about 10 to 17 percent above the estimated laboratory optimum. In addition, ground water was encountered at shallow depths. Excavations extending below grade several feet are likely to encounter over optimum to saturated soil conditions regardless of the time of year. Rubber-tire equipment should not be allowed in excavations deeper than 3 feet below grade. Contractors should be aware that operation of heavy grading equipment can destabilize wet clays. Consideration should be given to the use of light-weight equipment and sheepsfoot compactors to prepare the site subgrade.

There are several alternatives to facilitate fill placement and trench backfill if earthwork is performed during the wet winter season, and the moisture content of the fill materials increases significantly above optimum moisture.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content
- Install a geo-synthetic (geotextile or geogrid) to reduce surface yielding and reinforce soft fill
- Chemically treat with lime, kiln-dust, or cement to reduce the moisture content and increase the strength of the fill

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

5.8 Lime Treatment

Lime-treatment may be used to winterize and provide long-term support of building slab-on-grade and pavement areas at the site. In addition, lime-treated soil may be used as the required non-expansive fill (NEF) below interior slabs or exterior flatwork. Quicklime (CaO) is generally the most common type of lime application used in this area.

If desired, the existing subgrade may be treated with quicklime to a depth of 18 inches. The lime-treated soils should be compacted to at least 90 percent relative compaction. For the purpose of cost estimates, we recommend that 4 percent of quicklime by weight be assumed, based on a dry soil unit weight of 110 pounds per cubic foot. A final percentage of lime should be determined prior to construction using the existing soils and a sample of the actual quicklime source to be used.

The quicklime should be placed and mixed in accordance with Caltrans Standard Specifications, Chapter 24. We recommend mixing at least twice and that at least one mix occurs after the first mix has been allowed to cure overnight. Once the treated materials have broken down sufficiently, compaction of the approved treated materials may proceed. If the lime-treated soil will be used to satisfy the NEF requirement, confirmation PI tests should be performed with results of 15 or less. Compaction test results for treated soil will be available the day following testing because moisture contents of lime treated soil must be determined by oven drying. The surface of the lime-treated section should be firm and unyielding prior to placing the Class 2 aggregate base or crushed rock.

5.9 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to the levels noted in the "Compaction" section above by mechanical means only. Water jetting of trench backfill should not be allowed.

If ground water is encountered in utility trench excavations, temporary construction dewatering may be required and crushed rock may be needed to stabilize the trench bottom (if approved by the local jurisdiction) to provide a stable working surface for utility installation and backfill. Gravel may also be needed if the trench excavations are unstable. The crushed rock should extend up to above the ground water level. The crushed rock for stabilization and bedding should be compacted by vibratory methods until no further volume reduction is observed. Re-use of excavated soils may be difficult due to high in-situ moisture contents. If desired to re-use the excavated soils, drying of the soils would be required.

Utility trenches should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the foundation bearing surface to the bottom edge of the trench. This recommendation does not concern structures supported on deep foundations.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas and coming into contact with the expansive subgrade material.

5.10 Temporary Dewatering

Depending on the depths of utility and other excavations, temporary dewatering may be required. Temporary dewatering during construction should be the contractor's responsibility. The selection of equipment and methods of dewatering should be left up to the contractor since construction experience may determine which method is more economical and/or appropriate. The contractor should note that special considerations may be required prior to discharge of ground water from dewatering activities depending on the environmental impacts at the site or at nearby locations. These requirements may include storage and testing under permit prior to discharge. Impacted ground water may require discharge at an offsite facility.

5.11 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

5.12 Surface Drainage

Positive surface water drainage gradients (2 percent minimum in landscaped areas and 1 percent minimum in paved areas) should be provided adjacent to the building to direct surface water away from foundations and slabs towards suitable discharge

facilities. Ponding of surface water should not be allowed on or adjacent to the building, slabs-on-grade, or pavements. Roof runoff should be carried at least 5 feet away from foundations and slabs-on-grade in closed pipes and discharged to suitable facilities. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

5.13 Landscaping Considerations

As the near-surface soils are moderately to highly expansive, we recommend greatly restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low precipitation sprinkler heads,
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements.

We recommend that the landscape architect incorporate these items into the landscaping plans, and that we review the plans before construction.

5.14 Construction Observation

A representative from our company should observe and test the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including, site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule, we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

6.0 1997 UBC/2001 CBC SITE SEISMIC COEFFICIENTS

Chapter 16 of the 1997 Uniform Building Code (UBC) and 2001 California Building Code (CBC) describes the procedure for seismic design of the structure, which includes the seismic coefficients C_a and C_v . These coefficients are developed from parameters contained in series of tables and figures in the code. Section 1636.2 and Table 16-J describes the procedure for assigning a soil profile type (S_A through S_F) to the site.

Based on our borings, CPTs, and review of available alluvial thickness maps of Santa Clara County (Rogers and Williams, 1974), the site is underlain by stiff soils extending to depths on the order of 500 feet. Therefore, the site can be characterized as soil profile type S_D generally described as a stiff soil profile with average Standard Penetration Test (N) values in the range of 15 to 50 blows per foot. Based on the above information and local seismic sources, the site may be characterized for design based on Chapter 16 of the 1997 UBC and/or 2001 CBC using the information in Table 3 below.

Table 3. 1997 UBC/2001 CBC Site Categorization and Seismic Coefficients

Categorization/Coefficient	Design Value
Soil Profile Type (Table 16-J)	S_D
Seismic Zone (Figure 16-2)	4
Seismic Zone Factor (Table 16-I)	0.4
Seismic Source Name	Hayward (SE Extension)
Seismic Source Type (Table 16-U)	B
Distance to Seismic Source (kilometers)	3.1
Near Source Factor N_a (Table 16-S)	1.19
Near Source Factor N_v (Table 16-T)	1.45
Seismic Coefficient C_a (Table 16-Q)	0.52
Seismic Coefficient C_v (Table 16-R)	0.93

7.0 DEEP FOUNDATIONS

The site is underlain by moderately compressible clays located within shallow foundation influence zones. As discussed, we performed settlement analyses based on the structural loads provided by the project structural engineer for the three-elevated-level structure using an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus live loads. Our analyses indicate that total static settlement will be on the order of 1 to 2¼ inches. In addition, liquefaction-induced total settlements of about 3¾ to 7 inches are also anticipated following an earthquake or strong ground shaking. Therefore, without any liquefaction-induced settlements, estimated static differential settlement is on the order of 1-inch, which is about the limit of what shallow foundation systems consider tolerable. Added to the static settlements are liquefaction-induced settlements that ranged up to about 7 inches, which are not tolerable to shallow foundations. As discussed in the SCEC report, differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement. Based on this information, we estimate that the combined static and seismic differential settlements will be on the order of 2¼ to 4½ inches between columns.

Based on our conversations with the project structural engineer, the site is not suitable for shallow foundations due to significant static and seismic settlements. To reduce damage to the planned parking structure, we recommend that the parking structure be supported on deep foundations. In addition, considerations should be given to designing the garage concrete slab-on-grade as a structural slab. Recommendations for deep foundation alternatives and slab-on-grade are presented below.

7.1 Downdrag on Piles

The design issues involved with the deep foundation design included the generation of downdrag forces on the piles due to the magnitude of the estimated liquefaction-induced settlements. For downdrag to occur, it takes about $\frac{2}{3}$ to $\frac{3}{4}$ -inch of settlement within a layer. There are two sand strata present, generally from about 4 to 30 feet and 40 to 50 feet. In the Zone B area, the deeper sand layer is dense enough that the estimated liquefaction-induced settlements are less than $\frac{2}{3}$ -inch; either driven piles, or conventional or displacement augercast piles, all with downdrag to 28 feet incorporated in the length design, could be used. Where the deeper sand

layer is anticipated to cause downdrag (Zone C), a conventional driven pile or conventional non-displacement augercast pile would have to be about 105 feet long before starting to generate positive vertical capacity to support the structure loads, resulting in very long piles with very low capacity. For the Zone C area to use piles of similar length as those within Zone B, additional ground improvement (such as compaction grouting or ground improvement piles) would be required to improve the deeper layer; the upper sands have a higher fines content and will likely not be improved such that downdrag could be eliminated.

7.2 Driven Pre-Cast Concrete Piles

While driven piles are the most common pile choice in the area, we anticipate that pre-drilling to depths of about 30 to 50 feet may be required for the southern portion of the site due to thick sand layers that will be difficult to drive through. This will generate significant spoils for off-haul as well as extra drilling time. Please note that pea gravel may be encountered in the former UST backfill excavation areas, which may cause cave-ins during pile pre-drilling.

As previously discussed, maximum dead plus live column loads on the order of 210 to 920 kips for the three-elevated-level parking structure are expected. Since most of the site will likely experience down-drag due to liquefaction-induced settlement during an earthquake, we have divided the site into three zones as shown on Figure 2. Zone A does not need to be designed to resist down-drag as liquefaction is not anticipated; Zone B will need to be designed to resist down-drag to a depth of 28 feet; and Zone C will need to be designed to resist down-drag to a depth of 49 feet. We have computed vertical capacities for all zones and presented our results in Figure 4. The allowable capacities were computed based on a factor of safety of 2.0 for Zone A, where no downdrag is anticipated; for Zones B and C, the capacities were based on a full negative skin friction reduction factor of 0.85 and a factor of safety of 1.5 below the zone of downdrag.

Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities. The allowable downward and uplift capacities may be increased by one-third under transient loading, including wind and seismic. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile width.

As previously discussed, the former UST excavation areas were backfilled with undocumented fill to depths of about 9½ to 10 feet. Since the former UST excavation areas are located within the zones that will be potentially impacted by down-drag, the supporting capacity of undocumented fills that is typically neglected has been taken into account.

If desired to increase the driven pile capacity, compaction grouting to densify the potentially liquefiable sand layers between depths of 30 and 50 feet may be performed, eliminating the potential for down-drag. We recommend that the compaction grouting be performed after the piles were driven. Upon completion of the compaction grouting, we recommend that additional subsurface explorations be performed to confirm that the potentially liquefiable sand layers were adequately densified. If this option is desired, we should be consulted further.

7.2.1 Lateral Loads on Driven Piles

Lateral load resistance for pile-supported structures may be developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon many factors, including pile stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pile.

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. As discussed above, piles located within Zones B and C will likely experience down-drag during strong ground shaking. We have developed a subsurface profile for each zone (Zones A, B, and C). However, since the deeper liquefiable sand layer that could induce down-drag in Zone C is below the depth to zero shear force and moment, the results for Zones B and C are similar. Due to encountering the potentially liquefiable soil layer in CPT-7 at a shallower depth, we recommend that shear walls between Lines B and C and Columns 4 to 8 be designed using the parameters presented in the "B around CPT-7" in Table 4 below. The lateral load characteristics for 14-inch-square, driven concrete piles with free-head and fixed-head conditions are presented below.

Table 4. Estimated Lateral Pile Response – 14" Square Piles

Zone	Head Condition	Deflection (inches)	Maximum Shear Force (kips)	Maximum Moment (ft-kips)	Depth to Maximum Moment (ft)	Depth to Zero Moment (ft)
A	Free	¼	16	49	5½	14
		½	22	79	6	15
	Fixed	¼	33	125	Top of Pile	4½
		½	45	193		5
B and C	Free	¼	15	47	5½	14½
		½	20	75	6	15½
	Fixed	¼	30	120	Top of Pile	4½
		½	40	186		5
B around CPT-7	Free	¼	12	34	6	15
		½	17	52	7½	17.5
	Fixed	¼	22	85	Top of Pile	6
		½	30	133		7

The analysis results represent the probable response of the piles under short-term loading conditions and include no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. A pile stiffness of (EI) 1.4×10^{10} lb-in² has been assumed in our calculations of load deflection for the 14-inch piles. We assumed a minimum compressive strength of 6,000 pound per square inch for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). We should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

7.2.2 WEAP Analysis

At a minimum, we recommend that the pile contractor have a wave equation analysis of piles (WEAP) performed to confirm compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. We should review the WEAP results prior to mobilization of pile driving equipment to the site.

7.2.3 Indicator Piles

It has been our experience that uncertainties associated with production pile driving can be reduced considerably by implementing an indicator pile program. An indicator pile program will also provide a better means of confirming the limits of layers where high driving resistance may be encountered, and to more accurately estimate final pile lengths.

We recommend that at least eight indicator piles be installed before the final pile casting lengths have been selected. The indicator piles should be driven with the same equipment that will be used to drive the production piles. We should review or select the indicator pile locations when structural drawings are made available. The indicator pile cast lengths should be based on the design lengths required to meet the desired capacity, plus 5 feet. It is expected that some indicator piles may not be driven to their entire length and will require cutting to provide the desired butt elevation. Indicator piles can be used for support of the structure and, therefore, should be located appropriately. We also suggest that one or more spare piles be delivered to the site during the indicator program.

7.2.4 PDA Monitoring

If desired, a Pile Driving Analyzer (PDA) can be used during the indicator pile program to determine approximate pile capacities through dynamic testing. PDA monitoring may allow a reduction in production pile lengths and thus cost savings to the project. PDA monitoring should be performed during indicator driving and on selected piles for restrikes; restrikes should be performed no sooner than seven days after initial driving. *Please note that restrike testing more than one day after installation may significantly alter the contractor's sequencing. Therefore, if restrike testing is selected for this project, it should be clearly identified on the plans and specifications to avoid unexpected costly change-orders for out of sequence moves.* PDA monitoring would be especially beneficial for checking tensile stresses in the piles and for evaluating pile integrity on any piles suspected of being damaged during indicator or production driving. Piles designated for PDA monitoring during indicator pile installation should be at least 10 feet longer than design length so that the gauges are not driven into the ground.

7.2.5 Production Pile Installation

We recommend that a pile hammer capable of delivering a minimum rated driving energy of 60,000 foot-pounds be used. If indicator piles are installed, the same hammer should be used for both the indicator piles and the production piles. The pile contractor should perform wave equation analysis to confirm the compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. We should review the wave equation results prior to mobilization of pile driving equipment to the site.

Since the piles are designed for skin friction support, they should be driven to the desired tip elevation. If difficult driving conditions are encountered, we should review the driving record and evaluate potential tip capacity to allow reduction in pile length. We may also recommend that a Pile Driving Analyzer (PDA) be used during production driving to determine approximate pile capacities through dynamic analyses. PDA monitoring would be especially beneficial for checking restrike capacities of any piles short of required tip elevation or for evaluating pile integrity on any piles suspected of being damaged during driving. We should observe all indicator and production pile installation on a full-time basis.

7.3 Displacement Augercast Piles

While less common in the Bay area, augercast piles have been successfully used for projects in Milpitas and downtown San Jose in similar soil conditions. Augercast piles are cast-in-place concrete piles that are drilled using a hollow-stem auger and pumping sand-cement grout through the bottom of the auger as the auger is retracted. Two types of augercast piles are available: APG piles, which like piers, remove the soil column and replace it with grout; and the APGD piles, which displace the soil prior to grout placement. We anticipate that displacement piles are feasible so the drilling spoils generated would be minor. Augercast piles are a low noise and vibration installation compared to driven piles, and would not require pre-drilling through the thick sand layers. Various types of steel reinforcing including rebar cages or H-piles may be installed into the still wet grout after drilling.

We are currently in construction on a project in Milpitas, the Terra Serena Multi-Family Residential Development, where displacement augercast piles were used in conjunction with surrounding rings of non-structural displacement ground improvement piles to address similar geotechnical issues. The ground improvement piles were constructed using the same drilling rig but a sand cement slurry mix was placed in lieu of the structural sand cement grout, and no reinforcing was used. For the Terra Serena project we were able to demonstrate with field testing that the displacement method of installing the augercast piles was able to improve (increase the density) of sand layers with about 15 percent fines or less. So where we had deeper sand layers that would result in very long, low capacity piles, ground improvement piles were installed surrounding the structural pile groups. This provided a zone around the structural pile group of improved sand to either eliminate the potential for downdrag where most of the sand layers had less than about 15 percent fines or reduce the downdrag to shallower depths where the sands were dirtier and we were unable to demonstrate improvement.

A similar system could be constructed using pre-cast piles as ground improvement piles surrounding the structural pile groups; however, it would likely be more costly,

and if reinforcing wasn't cast into the piles, there could be breakage due to driving stresses. Ground improvement techniques such as stone columns or compaction grouting could also be used in conjunction with driven piles; however, this would require two construction company mobilizations, and the piles would have to be pre-drilled and driven through the improved ground area. Improving the ground after pile driving could damage the pile. In our opinion, displacement augercast piles with non-structural ground improvement piles surrounding the structural groups will likely be the most cost-effective foundation alternative.

While conventional augercast piling has been used for decades, displacement augercast piling was only developed within the last decade, and we understand that its use is relatively new to the Bay area. If a better idea of the system is desired, we suggest that Berkel & Company be asked to provide a presentation of the technique and to answer questions that may come up. Due to the variability of the sand layers and the cost implications associated with added ground improvement piles, confirmation CPT testing will be required to evaluate the effectiveness of the non-structural pile's ground improvement and adjust the spacing, if necessary. This should be performed after demolition is complete and preferably at the same time as the construction of the test piles. A member of our staff should review all CPT test results prior to acceptance of performance.

7.3.1 Vertical Loads

We also contacted Berkel & Company, a licensed Augered Cast-in-Place Piles design-builder, to evaluate load capacities. Figure 4 presents allowable vertical capacities for 16- and 18-inch augercast piles as well as 14-inch precast driven piles. Our exploration indicates there is no significantly thick or continuous dense sand layer that would provide end-bearing support; therefore, pile support is expected to come predominantly from frictional support in the stiff clays. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities. The pile capacities and uplift loads may be increased by one-third under transient loading, including wind and seismic.

The capacities in Zone C (as depicted on the Site Plan, Figure 2) assume that 45- to 50-foot ground improvement piles will be installed surrounding the structural piles/pile groups, allowing down drag to be included only to a depth of 28 feet.. The improvement piles will need to be at least 45 to 50 feet long to penetrate the liquefiable layers. Zones A and B will not have ground improvement piles.

We have assumed a base of pile cap at 3 feet below finished floor for our analysis. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile diameter.

Based on the maximum allowable loads for a single pile, we estimate total settlements of less than 1/2-inch to mobilize allowable static capacities. Therefore, post-construction pile foundation settlements of less than 1/4-inch should be considered.

7.3.2 Lateral Loads on Augercast Piles

Lateral load resistance for pile-supported structures may be developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent

upon many factors, including pile stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pile.

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. Since the liquefiable sand layers in Zones B and C are recommended to be improved with non-structural piles, the results for all zones are similar. Due to encountering the potentially liquefiable soil layer in CPT-7 at a shallower depth, we recommend that shear walls between Lines B and C and Columns 4 to 8 be designed using the parameters presented in the "B around CPT-7" in Table 5 below. The lateral load characteristics for 16-inch-diameter, augercast piles with free head and fixed head conditions are presented below.

Table 5. Estimated Lateral Pile Response – 16" Round Piles

Zone	Head Condition	Deflection (inches)	Maximum Shear Force (kips)	Maximum Moment (ft-kips)	Depth to Maximum Moment (ft)	Depth to Zero Moment (ft)
A	Free	¼	16	46	5	13
		½	22	72	5½	14½
	Fixed	¼	34	117	At pile cap	4½
		½	46	181		5
B and C	Free	¼	16	46	5	13
		½	22	72	5½	14½
	Fixed	¼	34	117	At pile cap	4½
		½	46	181		5
B around CPT-7	Free	¼	12	32	6	15
		½	17	48	7	17½
	Fixed	¼	22	80	At pile cap	5½
		½	30	125		6

The analysis results represent the probable response of the piles under short-term loading conditions and include no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. A pile stiffness (EI) of 1.2×10^{10} lb-in² has been assumed in our calculations of load deflection. We assumed a minimum compressive strength of 4,000 pound per square inch for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). We should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

7.3.3 Pile Load Tests

Load testing for augercast pile foundations typically consists of performing at least one static load test. We recommend at least two load tests for the proposed parking garage. Static load tests include installing a test pile either in a production location or not, with four surrounding anchor piles supporting a load frame to resist jacking against the test pile. During installation of the test piles, the contractor should allow for monitoring pile displacement at the top of pile, middle, and pile tip. Monitoring can be by strain gauges or capped conduits placed in the pile, allowing telltales to be placed during testing. This will allow for observation of the loads at which the skin friction is mobilized. A more detailed description of static load tests is presented in ASTM D5780. A member of our staff should be present during installation of the test piles and load testing, and have the opportunity to review the test results.

Alternatively, the contractor may mobilize pile-driving equipment to perform PDA testing as discussed in the driven pile section above.

7.4 Interior Slabs-On-Grade

Due to the high expansion potential of the surficial soils, we recommend that any interior slabs-on-grade for any retail/commercial areas be supported on at least 24 inches of non-expansive fill (NEF) to reduce the likelihood of slab damage from heave. Garage slab-on-grade pavements should be at least 5 inches thick and supported on at least 18 inches of NEF. The upper 6 inches should consist of either Class 2 aggregate base or $\frac{3}{4}$ inch crushed rock. Alternatively, the garage slab may be supported on an 18-inch-deep lime-treated soil subgrade using quicklime (CaO), eliminating the rock requirement. Typically, however, 2 to 3 inches of Class 2 aggregate base is placed at the top of a lime-treated pad for constructability during winter months. An 18-inch-thick lime-treated pad may also satisfy the lower 18 inches of the 24-inch-thick NEF requirement if the lower level slab is the banquet room slab. Please refer to the "Earthwork" section for a discussion of estimated quicklime percentages. We should perform laboratory testing on the lime-treated soil to confirm that the PI is 15 or less. The garage slab should have a minimum concrete compressive strength of 3,000 pounds per square inch (psi).

If desired, the garage slab may be designed as a structural slab to accommodate liquefaction-induced differential movement. We recommend at a minimum, the elevator pit and equipment room slabs be structurally supported. The structural engineer should determine the structural slab thickness in accordance with the estimated liquefaction-induced settlements as discussed in the "Liquefaction" section of this report. The structural slab should also be supported on at least 12 inches of NEF. The upper 6 inches should consist of either Class 2 aggregate base or $\frac{3}{4}$ -inch crushed rock. The aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). The crushed rock should be consolidated in-place to provide firm, uniform support for the slab. The garage slab should have a minimum concrete compressive strength of 3,000 psi. Prior to placing the aggregate base or crushed rock, the subgrade should be prepared in accordance with the recommendations presented in the "Subgrade Preparation" section of this report.

We also recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the floor slab has been

cast. Before slab construction, the subgrade surface should be proof-rolled to provide a smooth and firm surface for slab support.

Post-construction cracking of concrete slabs-on-grade is inherent in any project, especially where soil expansion potential is high. In our opinion, consideration should be given toward a maximum control joint spacing of 8 to 10 feet in both directions for the interior slab-on-grade construction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

If desired to limit moisture rise through slab-on-grade floors, the guidelines presented in the "Moisture Protection Considerations" section of this report should be considered.

7.5 Building Pad Moisture Conditioning

Due to the high expansion potential of the surficial soils, we recommend that finished subgrade be moisture conditioned to at least 3 percent over optimum in the upper 12 inches of the pad subgrade prior to placing the moisture barrier system. The moisture content of the finished subgrade should be checked within 24 hours prior to the construction of the moisture barrier.

7.6 Moisture Protection Considerations

Since the long-term performance of concrete slabs-on-grade depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete slab of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with slab-on-grade construction. Moisture barriers are not intended for the garage slab areas unless desired by the owner. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 10-mil thick vapor barrier meeting minimum ASTM E 1745, Class C requirements should be placed directly below the slab (no sand). The vapor barrier should extend to the edge of the slab. If the slab is 8 inches thick or less, at least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break. The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations. The crushed rock may be considered as the upper 4 inches of the non-expansive fill requirement for slab-on-grade construction.
- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.
- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels should not be permitted.

- All concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.
- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. The application of these guidelines does not affect the geotechnical aspects of the foundation performance.

8.0 AT-GRADE SITE RETAINING WALLS

8.1 Lateral Earth Pressures

Any proposed at-grade site retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of $8H$ pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

We understand that the elevator shafts will be supported by the pile foundation system; however, we recommend the elevator retaining walls should be designed as undrained conditions and water-proofed due to the shallow ground water conditions.

8.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and

to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively impervious compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

8.3 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 87 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

8.4 Foundation

At-grade retaining walls not supported by the parking structure foundations may be supported on a continuous spread footing bearing on natural, undisturbed soil or compacted fill. All footings should have a minimum width of 12 inches and footing bottoms should extend at least 24 inches below lowest adjacent finished grade. Because of the high expansion potential of the near-surface soils, this relatively deeper footing is recommended to place bearing surfaces below the zone of significant moisture fluctuation in order to reduce the effects of heave or shrinkage.

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. It is essential that we observe all footing excavations before reinforcing steel is placed.

Due to the anticipated relatively light loading of any site retaining walls, we estimate that total settlement is less than ½-inch due to static loads. As discussed in the "Liquefaction" section above, several silt and sand layers may theoretically liquefy,

resulting in about 3¾ to 7 inches of total settlement. As discussed in the SCEC report, differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement. Therefore, we estimate that an additional of 1¾ to 3½ inches of liquefaction-induced differential settlement over a lateral distance of 50 feet is possible following an earthquake or strong ground shaking.

8.5 Lateral Loads

Lateral loads may be resisted by friction between the bottom of the footings and the supporting subgrade. An ultimate frictional resistance of 0.35 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against foundations poured neat against competent soil. We recommend that an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf be used in design. The structural engineer should apply an appropriate factor of safety to the above values. The upper 12 inches of soil should be neglected when calculating the lateral passive resistance unless covered by concrete slabs or pavements.

9.0 AT-GRADE PAVEMENTS

9.1 Asphalt Concrete

Based on our review of geotechnical reports previously prepared for adjacent sites and our engineering experience with the surficial soils in the site area, we judged an R-value of 5 to be applicable for design. Using estimated traffic indices for various pavement-loading requirements, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 6.

**Table 6. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R-Value = 5**

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	7.5	10.0
	4.5	2.5	9.5	12.0
Automobile Parking Channel	5.0	3.0	10.0	13.0
	5.5	3.0	12.0	15.0
Truck Access & Parking Areas	6.0	3.5	12.5	16.0
	6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Because the native soils at the site are highly expansive, some increased maintenance and reduction in pavement life can be expected. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

In addition, it has been our experience that asphalt concrete pavements constructed over expansive soils and adjacent to non-irrigated open space areas may experience cracking parallel to the edge of the pavement. This is typically caused by seasonal shrinkage and swelling adjacent to non-irrigated edges of the pavement. The cracks typically occur within the first few years of construction, and are typically located within a few to several feet of the edge of the pavement. The cracks, if they occur, can be filled with a bituminous sealant. Otherwise, a moisture barrier would need to be installed to a depth of at least 24 inches to reduce the potential for shrinkage of the pavement subgrade soils.

9.2 Exterior Portland Cement Concrete

Recommendations for exterior Portland Cement Concrete (PCC) pavements are presented below in Table 7. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 7. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
0.8	5
13	5½
130	6

Our design is based on an R-value of 5 and a 28-day unconfined compressive strength for concrete of at least 3,500 psi. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 12 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

9.3 Concrete Pavers

Where PCC paving blocks are planned for entranceways, we recommend that pavers be underlain by at least 18 inches of Class 2 aggregate base. For vehicular parking areas, pavers should be underlain by at least 15 inches of aggregate base. Our recommendations are based on the Portland Cement Association (1984) design criteria, which assumes a typical Equivalent Single Axle Load (ESAL) of 400,000 for entranceways and 100,000 for driveways and automobile parking areas. In addition, our design is based on the assumption that pavers are placed on a uniform bed of sand (no more than 1 inch thick) and have sufficient edge restraint such as provided by concrete curbs or border strips. Since pavers do not prevent water from entering into the aggregate base and underlying subgrade, the total aggregate base thickness is usually designed to be greater than that of an equivalent asphalt concrete pavement section.

If desired to increase the paver design life and decrease the potential for any long-term settlement, it would be beneficial to reinforce the subgrade with either a geotextile fabric or polymer geogrids. A geotextile fabric such as Mirafi 600X (or equivalent) or polymer geogrid such as Tensar BX1100 or BX1200 (or equivalent) would be appropriate. Polymer geogrids have a higher tensile strength and usually provide greater subgrade reinforcement than geotextile fabric. Alternatively, the pavers may be supported on a concrete sub-slab at least 6 inches thick overlying at least 12 inches of Class 2 aggregate base.

In addition, the 1-inch leveling course placed beneath pavers may shift after construction. It would therefore be beneficial to check that no more than approximately 1-inch of sand is placed during construction.

9.4 Pavement Cutoff

Because the native soils at the site are moderately to highly expansive, surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, "Deep-Root Moisture Barrier," or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated and should extend to a depth of at least 6 inches below the base rock layer.

9.5 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

9.6 Exterior Concrete Flatwork

Exterior concrete flatwork and sidewalks should be supported on at least 18 inches of non-expansive fill (NEF). If the subgrade soils will be treated with quicklime (CaO), it may satisfy as the NEF requirement. We should perform laboratory testing on this lime-treated soil to confirm that the PI is 15 or less. We recommend that the upper 4 inches of NEF should consist of Class 2 aggregate base compacted to at least 90 percent relative compaction (ASTM Test Method D1557, latest edition). Concrete sidewalks should be at least 4 inches thick. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Exterior Portland Cement Concrete" section of this report. Sidewalks in public right-of-way should be design in accordance with the City of Milpitas standards.

There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork and sidewalks. We are providing recommendations to reduce distress to concrete flatwork and sidewalks that includes moisture conditioning the subgrade soils, using non-expansive fill as mentioned above, and providing adequate construction and control joints to control cracks that do occur.

1. The minimum recommendation for concrete flatwork and sidewalks constructed on expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted, using moderate compaction effort, to a relative compaction between 87 to 92 percent (ASTM Test Method D1557, latest edition). In general, the subgrade should be relatively firm and non-yielding prior to construction.
2. Use a maximum control joint spacing of no more than 8 feet in each direction. Construction joints that abut the foundations or garage slabs should include a felt strip or approved equivalent that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At your option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel.

10.0 LIMITATIONS

This report has been prepared for the sole use of Chong Partners Architecture specifically for design and construction of the Midtown East Parking Structure project in Milpitas, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated locations, visual observations from our site reconnaissance, and review of other geotechnical data performed at the site provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings and CPTs do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC Lowney cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or

misinterpretation of TRC Lowney's report by others. Furthermore, TRC Lowney will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

11.0 REFERENCES

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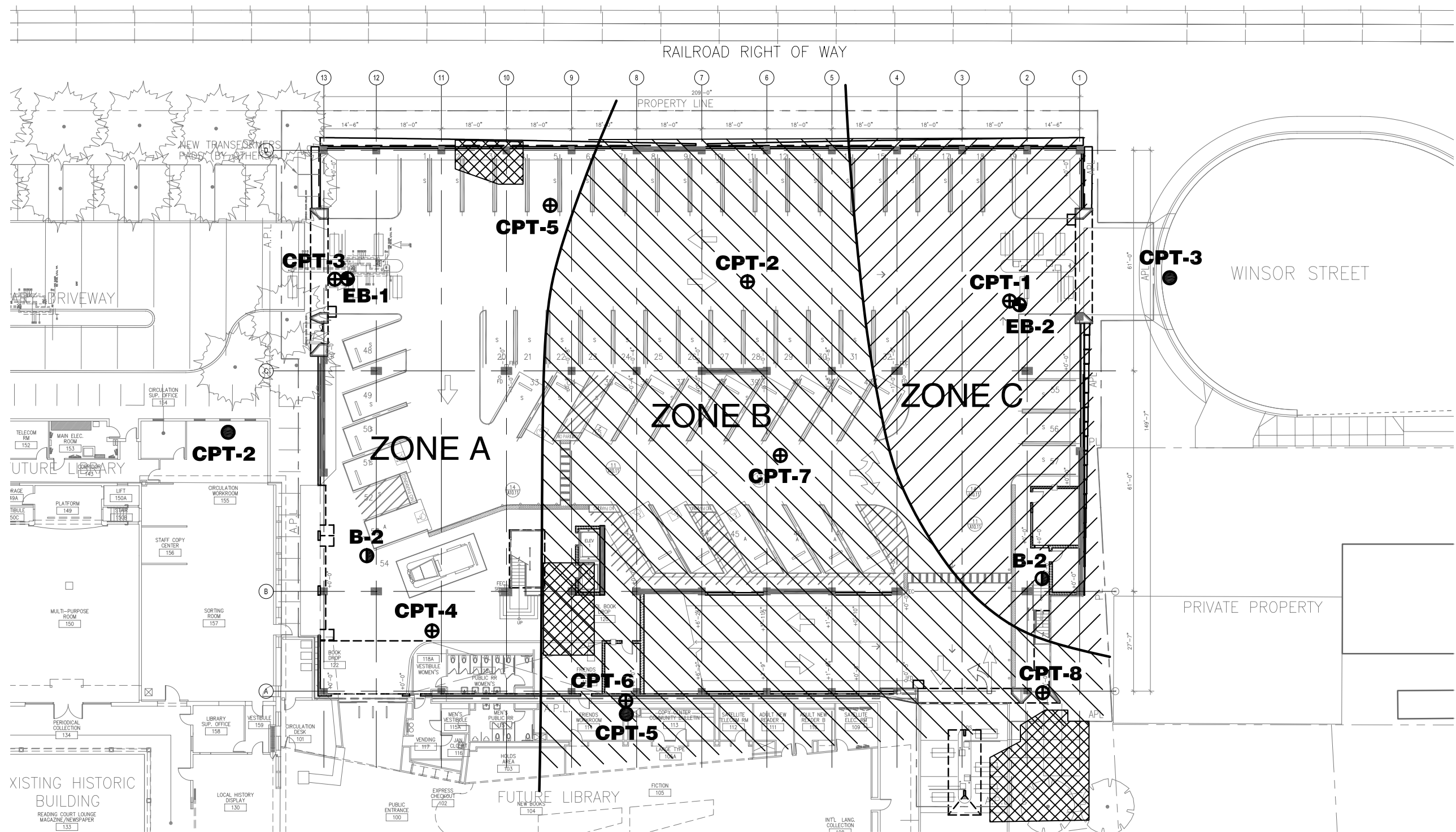
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* * * * *



LEGEND

- ⊕ - Approximate location of exploratory boring
- ⊕ - Approximate location of cone penetration test
- ① - Approximate location of exploratory boring by Treadwell & Rollo (2004)
- - Approximate location of cone penetration test by Treadwell & Rollo (2004)
- ▨ - Approximate location of former UST excavation
- ▨ - Approximate area potentially impacted by down-drag to a depth of 28 feet
- ▨ - Approximate area potentially impacted by down-drag to a depth of 49 feet



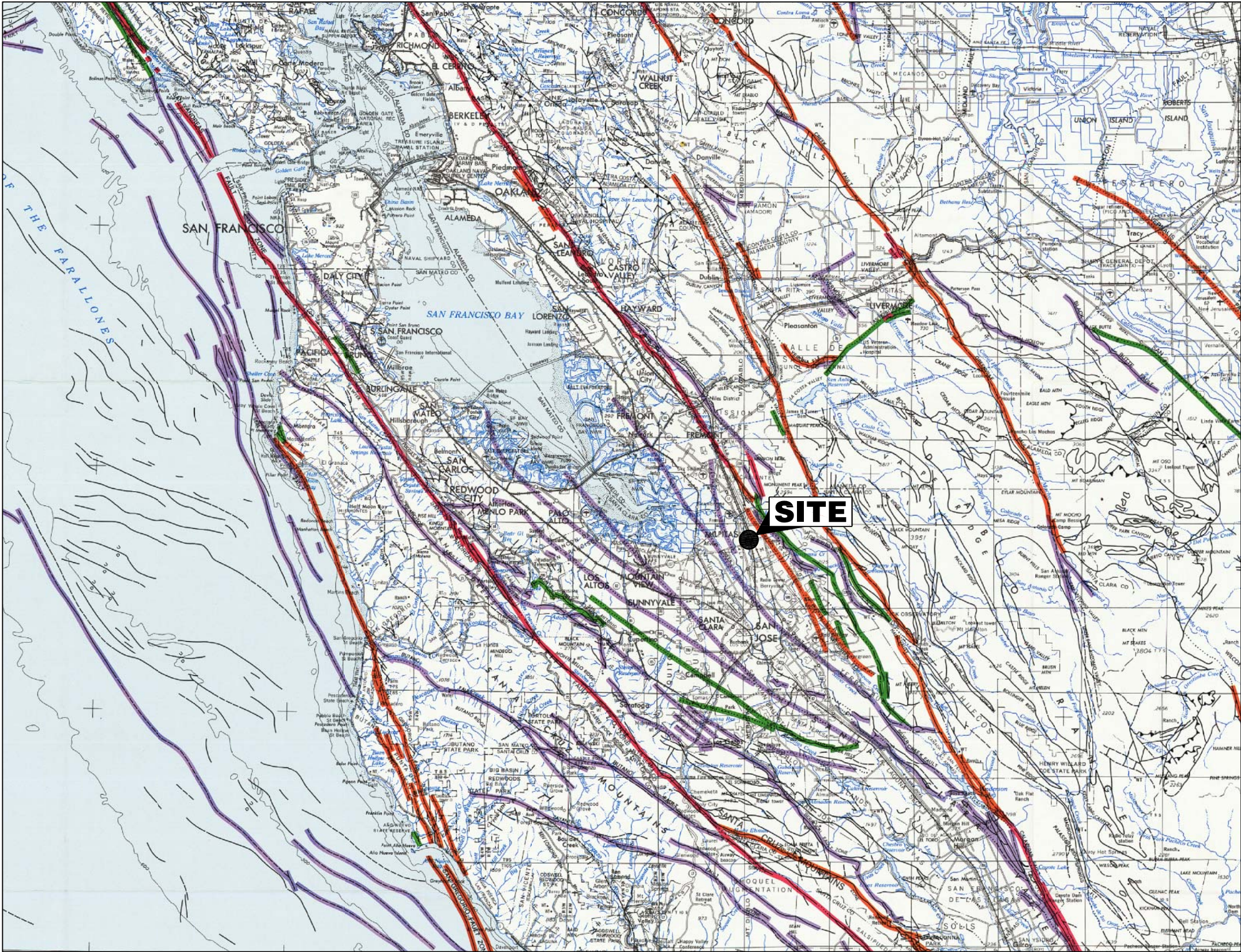
SITE PLAN
MIDTOWN EAST PARKING STRUCTURE
Mipitas, California

TRC Lowney

FIGURE 2

869-7A

Base by Unknown.



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

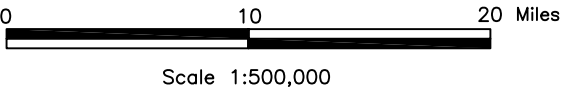
Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore ¹	DESCRIPTION
Quaternary	Late Quaternary	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		10,000			Displacement during Holocene time. ²
	Early Quaternary	700,000			Faults showing evidence of displacement during late Quaternary time. ³⁻⁴
Pre-Quaternary	Pleistocene	2,000,000			Quaternary (undifferentiated) faults – most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
	Miocene	5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

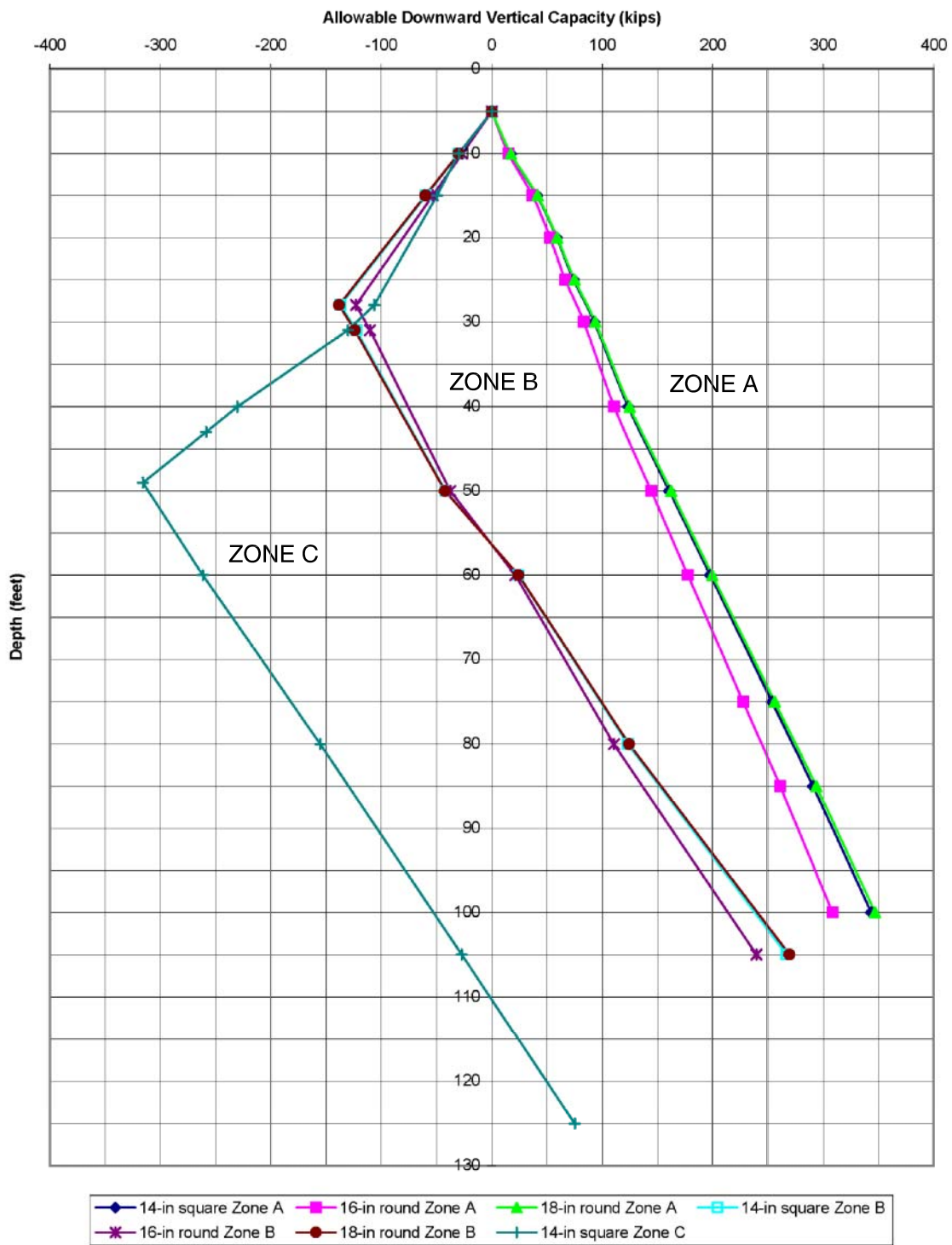


REGIONAL FAULT MAP
MIDTOWN EAST PARKING STRUCTURE
Milpitas, California

TRC Lowney

FIGURE 3

869-7A



Notes:

- 1.) Pile capacities for 14-inch square and 18-inch round piles are approximately the same values in each zone.
- 2.) Zone A are capacities with no liquefaction.
- 3.) Zone B are capacities with liquefaction to 28 feet, or Zone C piles with ground improvement to 50 feet.
- 4.) Zone C are capacities with liquefaction to 49 feet and no ground improvements.

1/06/EB

VERTICAL PILE CAPACITY
MIDTOWN EAST PARKING STRUCTURE
 Milpitas, California

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, rotary-wash and hollow-stem auger drilling, and Cone Penetration Test (CPT) equipment. One 8-inch-diameter hollow-stem boring was drilled on May 20, 2005 to a maximum depth of 45 feet; one 6-inch-diameter rotary-wash boring was drilled on May 16, 2005 to a maximum depth of 100 feet; eight CPTs were advanced on May 11, 2005 and January 20, 2006 to a maximum depth of 100 feet. CPT data was obtained at 0.16 feet intervals, and consisted of cone tip resistance, sleeve friction and other parameters. The data obtained was correlated using the references cited to determine the indicated soil type, shear strength, equivalent Standard Penetration Test (SPT), N-value (blows per foot), and other parameters. The approximate locations of the exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings and CPTs, as well as a key to the classification of the soil and CPT interpretations, are included as part of this appendix.

The locations of borings and CPTs were approximately determined by portable Global Positioning Systems (GPS) hand-held equipment and pacing from existing site boundaries for references. Elevations of the borings were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 2.5-inch I.D. samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube samplers, which were hydraulically pushed. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * *

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW		Well graded gravels, gravel-sand mixtures, little or no fines
			GP		Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM		Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC		Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW		Well graded sands, gravelly sands, little or no fines
			SP		Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM		Silty sands, sand-silt-mixtures, non-plastic fines
			SC		Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL		Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH		Inorganic clays of high plasticity, fat clays
			OH		Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT		Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE				CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	3/4"	3"	12"	
SILTS AND CLAY		SAND			GRAVEL		COBBLES
		FINE	MEDIUM	COARSE	FINE	COARSE	
	0.08	0.4	2	5	19	76mm	BOULDERS

GRAIN SIZES

	TERZAGHI SPLIT SPOON STANDARD PENETRATION		MODIFIED CALIFORNIA		ROCK CORE		PITCHER TUBE		NO RECOVERY
--	---	--	---------------------	--	-----------	--	--------------	--	-------------

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

EXPLORATORY BORING: EB-1

Sheet 1 of 4

DRILL RIG: FAILING 1500

BORING TYPE: ROTARY WASH

LOGGED BY: NB

START DATE: 5-16-05

FINISH DATE: 5-16-05

PROJECT NO: 869-7A

PROJECT: MIDTOWN EAST PARKING STRUCTURE

LOCATION: MILPITAS, CA

COMPLETION DEPTH: 100.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

asphalt concrete over aggregate base

FAT CLAY (CH)

stiff, moist, dark gray-brown, high plasticity
Plasticity Index = 41, Liquid Limit = 65

becomes dark gray

LEAN CLAY (CL)

medium stiff to stiff, moist, brown, some fine sand, low to moderate plasticity

LEAN CLAY (CL)

very stiff, moist, dark brown, moderate plasticity

FAT CLAY (CH)

stiff, moist, brown, trace fine sand, moderate to high plasticity

very stiff

Continued Next Page

SOIL TYPE	PENETRATION RESISTANCE (BLOW/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
						<div>○ Pocket Penetrometer</div> <div>△ Torvane</div> <div>● Unconfined Compression</div> <div>▲ U-U Triaxial Compression</div>
						1.0 2.0 3.0 4.0
CH	17		25	96		○
CH	18		32	90		○
	200psi					
CL	18		20	107		○
CL	110psi		30	93		▲
CL	74		20	113		△ ○
CH	280psi		21	105		▲
	45		23	103		△ ○

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,094

Easting: 6,153,296

LA. CORP. GDT 1/27/06 MV- GAR

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 4

DRILL RIG: FAILING 1500

BORING TYPE: ROTARY WASH

LOGGED BY: NB

START DATE: 5-16-05

FINISH DATE: 5-16-05

PROJECT NO: 869-7A

PROJECT: MIDTOWN EAST PARKING STRUCTURE

LOCATION: MILPITAS, CA

COMPLETION DEPTH: 100.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

Undrained Shear Strength (ksf)

- Pocket Penetrometer
- △ Torvane
- Unconfined Compression
- ▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30	FAT CLAY (CH) stiff, moist, brown, trace fine sand, moderate to high plasticity							
	35		CH	225psi		21	102		
	40	SANDY LEAN CLAY (CL) stiff, moist, brown, fine sand, low plasticity	CL	27		19	111	61	
	45	LEAN CLAY (CL) hard, moist, olive-gray, some fine sand, moderate plasticity	CL	50		19	108		
	50	SANDY LEAN CLAY (CL) medium stiff to stiff, moist, olive-brown, fine sand, low plasticity		29		20	108		
	55		CL	43		21	108		
	60	very stiff				22	104		

Continued Next Page

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,094

Easting: 6,153,296

LA CORP GDT 1/27/06 MV* GAR

EXPLORATORY BORING: EB-1 Cont'd

Sheet 3 of 4

DRILL RIG: FAILING 1500

BORING TYPE: ROTARY WASH

LOGGED BY: NB

START DATE: 5-16-05

FINISH DATE: 5-16-05

PROJECT NO: 869-7A

PROJECT: MIDTOWN EAST PARKING STRUCTURE

LOCATION: MILPITAS, CA

COMPLETION DEPTH: 100.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

ELEVATION (FT)

DEPTH (FT)

SOIL LEGEND

60

65

70

75

80

85

90

SANDY LEAN CLAY (CL)

medium stiff to stiff, moist, olive-brown, fine sand, low plasticity

CL

230psi

20

101

LEAN CLAY (CL)

very stiff, moist, brown to olivebrown, some fine sand, moderate plasticity

CL

70

20

101

52

23

101

FAT CLAY (CH)

very stiff, moist, olive-gray, trace fine sand, moderate to high plasticity

CH

74

26

101

280psi

71

24

98

LEAN CLAY (CL)

very stiff, moist, olive brown, some fine sand, moderate plasticity

CL

55

24

100

Continued Next Page

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,094

Easting: 6,153,296

LA CORP GDT 1/27/06 MV* GAR

EXPLORATORY BORING: EB-1 Cont'd

Sheet 4 of 4

DRILL RIG: FAILING 1500

BORING TYPE: ROTARY WASH

LOGGED BY: NB

START DATE: 5-16-05

FINISH DATE: 5-16-05

PROJECT NO: 869-7A

PROJECT: MIDTOWN EAST PARKING STRUCTURE

LOCATION: MILPITAS, CA

COMPLETION DEPTH: 100.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

LEAN CLAY (CL)

very stiff, moist, olive brown, some fine sand, moderate plasticity

Bottom of Boring at 100 feet

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)			
									○ Pocket Penetrometer	△ Torvane	● Unconfined Compression	▲ U-U Triaxial Compression
90												
95			CL	79	✕	18	111					
100				68	✕	19	110					
105												
110												
115												
120												

GROUND WATER OBSERVATIONS:

NOT APPLICABLE DUE TO ROTARY WASH CIRCULATION

Northing: 1,983,094

Easting: 6,153,296

LA CORP GDT 1/27/05 MV* GAR

EXPLORATORY BORING: EB-2

Sheet 1 of 2

DRILL RIG: FAILING 1500

BORING TYPE:

LOGGED BY: JH

START DATE: 5-20-05

FINISH DATE: 5-20-05

PROJECT NO: 869-7A

PROJECT: MIDTOWN EAST PARKING STRUCTURE

LOCATION: MILPITAS, CA

COMPLETION DEPTH: 43.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	0		4 inches asphalt concrete over 10 inches aggregate base							
	0		FAT CLAY (CH) very stiff, moist, dark gray, trace fine sand, high plasticity		30		23	96		
	2		stiff		22		30	88		
	5		medium stiff	CH	15		33	87		
	10		LEAN CLAY (CL) stiff to very stiff, moist, brown, trace fine sand, low to moderate plasticity	CL	23		21	107		
	15		SILTY, CLAYEY SAND (SC-SM) loose, moist, brown, fine sand Plasticity Index = 6, Liquid Limit = 25	SC-SM	8		22	44		
	16				6		25	33		
	17		SILTY SAND (SM) medium dense, moist, brown, fine sand, non plastic fines		11		19	33		
	18				12					
	20			SM	14		27	35		
	22				22		25	33		
	25				15					
	26				15		26	17		
	28		dense, fine to coarse sand, some fine gravel		44					
	29		POORLY GRADED SAND WITH SILT (SP-SM) very dense, moist, gray, fine to coarse sand, some fine gravel	SP-SM	50/6"		14	9		
	30				77					

Continued Next Page

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 10.8 FEET

Northing: 1,982,922

Easting: 6,153,328

LA CORP.GDT 1/27/06 MV* GAR

EXPLORATORY BORING: EB-2 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 1500

BORING TYPE:

LOGGED BY: JH

START DATE: 5-20-05

FINISH DATE: 5-20-05

PROJECT NO: 869-7A

PROJECT: MIDTOWN EAST PARKING STRUCTURE

LOCATION: MILPITAS, CA

COMPLETION DEPTH: 43.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	30			SP-SM						
	32		LEAN CLAY (CL) very stiff, moist, brown, some fine sand, moderate plasticity	CL	12					
	34			CL	47		19	111		
	36									
	38		SILTY SAND (SM) medium dense, moist, brown, fine to medium sand	SM						
	40			SM	30		17		34	
	42		very dense, decreasing silt							
	44				50/3"		23		13	
	46		Bottom of Boring at 43½ feet							
	48									
	50									
	52									
	54									
	56									
	58									
	60									

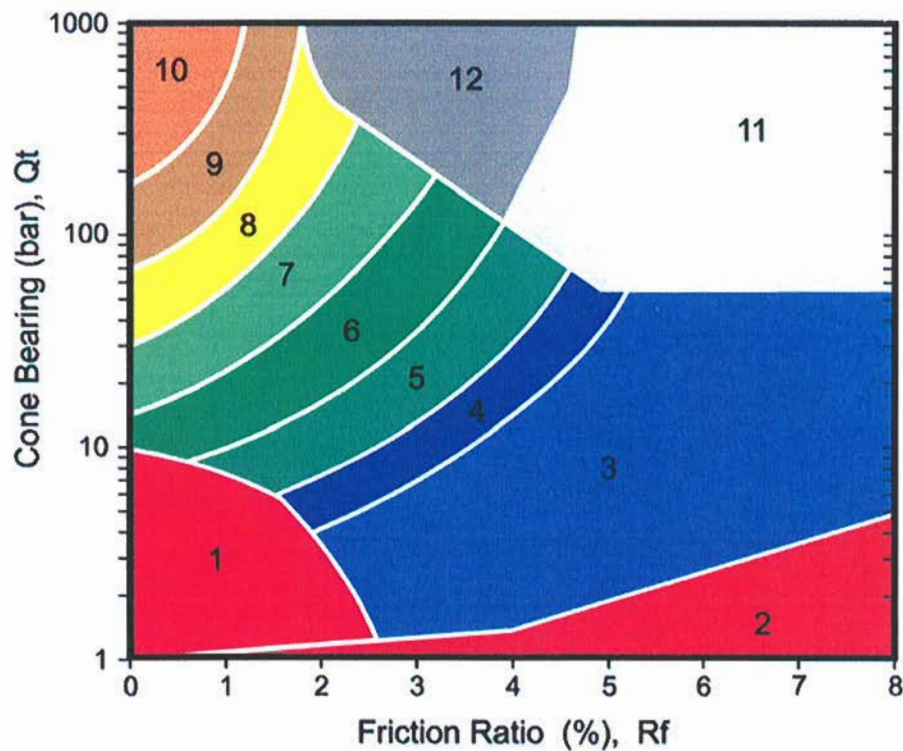
GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 10.8 FEET

Northing: 1,982,922

Easting: 6,153,328

LA CORP.GDT 1/27/06 MV* GAR

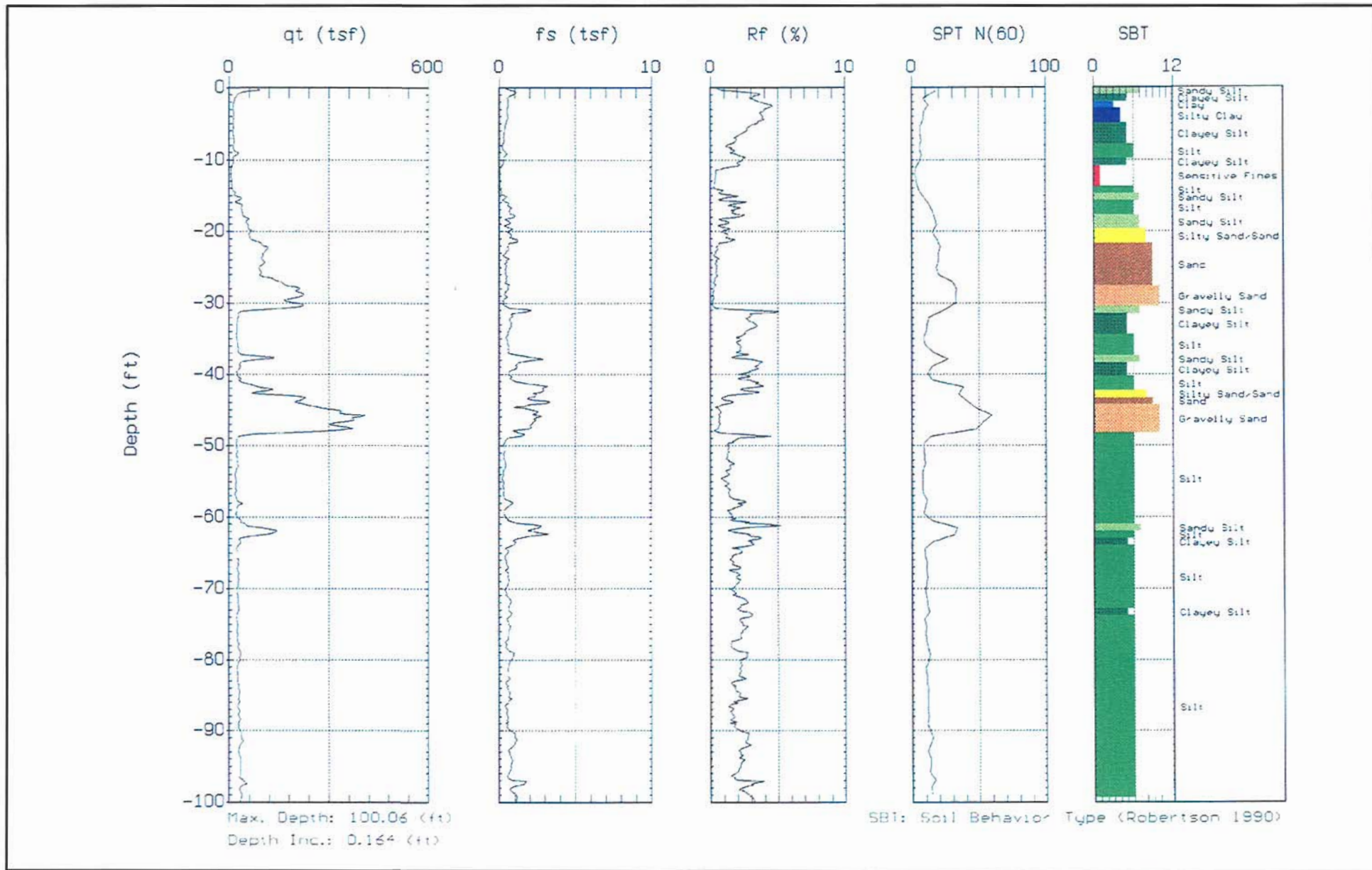


Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

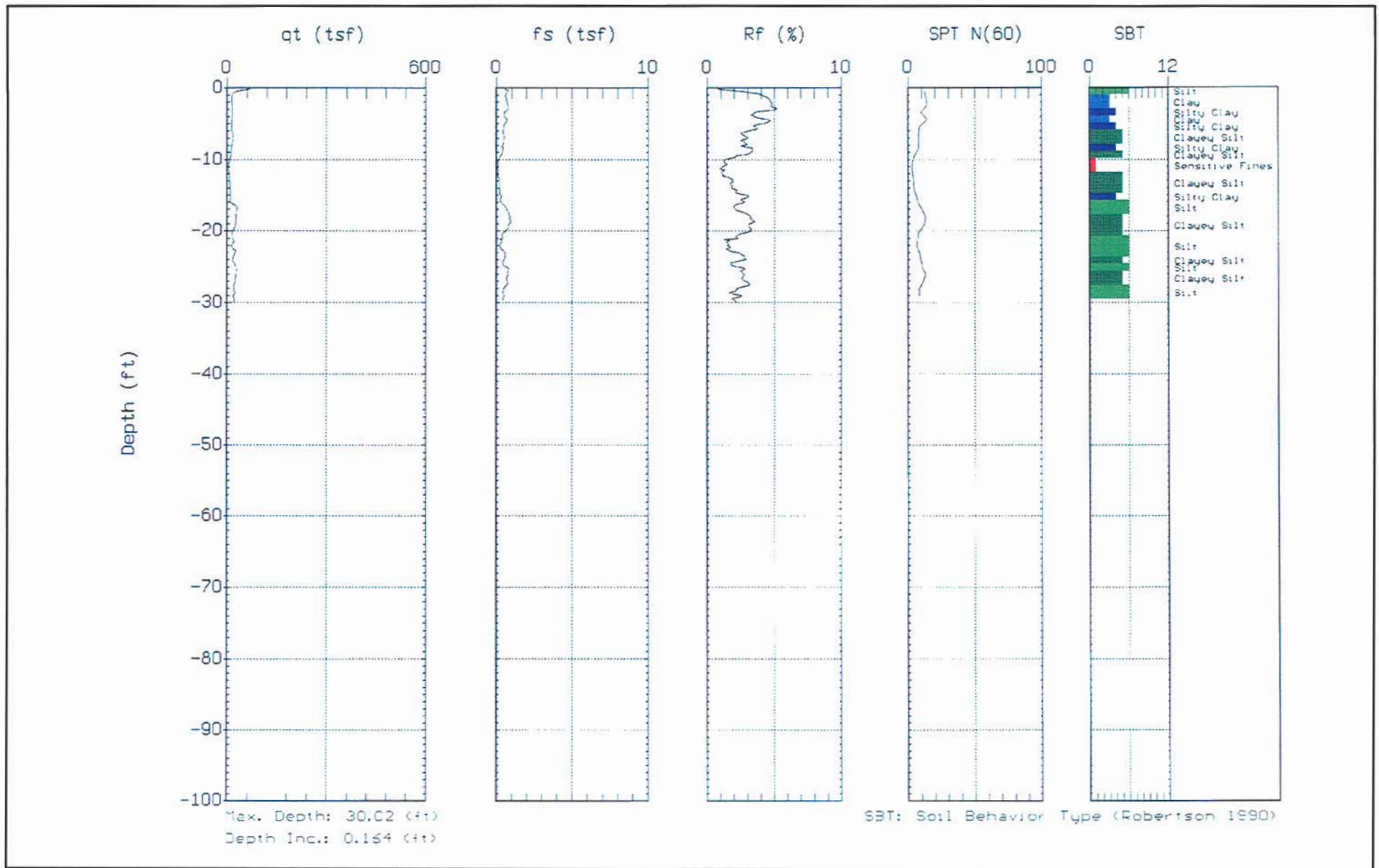
Robertson (1990)

KEY TO CONE PENETROMETER TEST



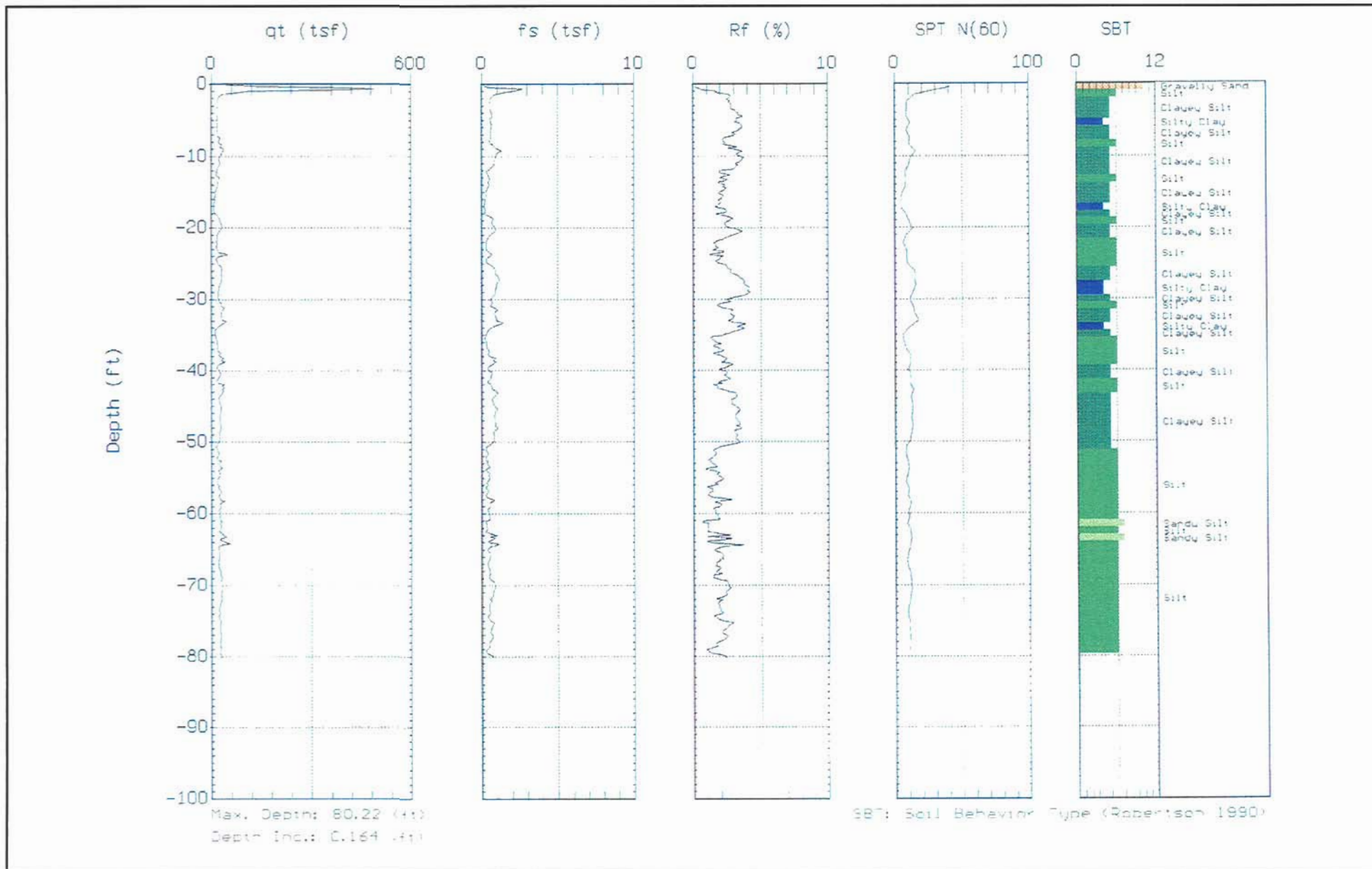
6/05*EB

CONE PENETRATION TEST - CPT-2



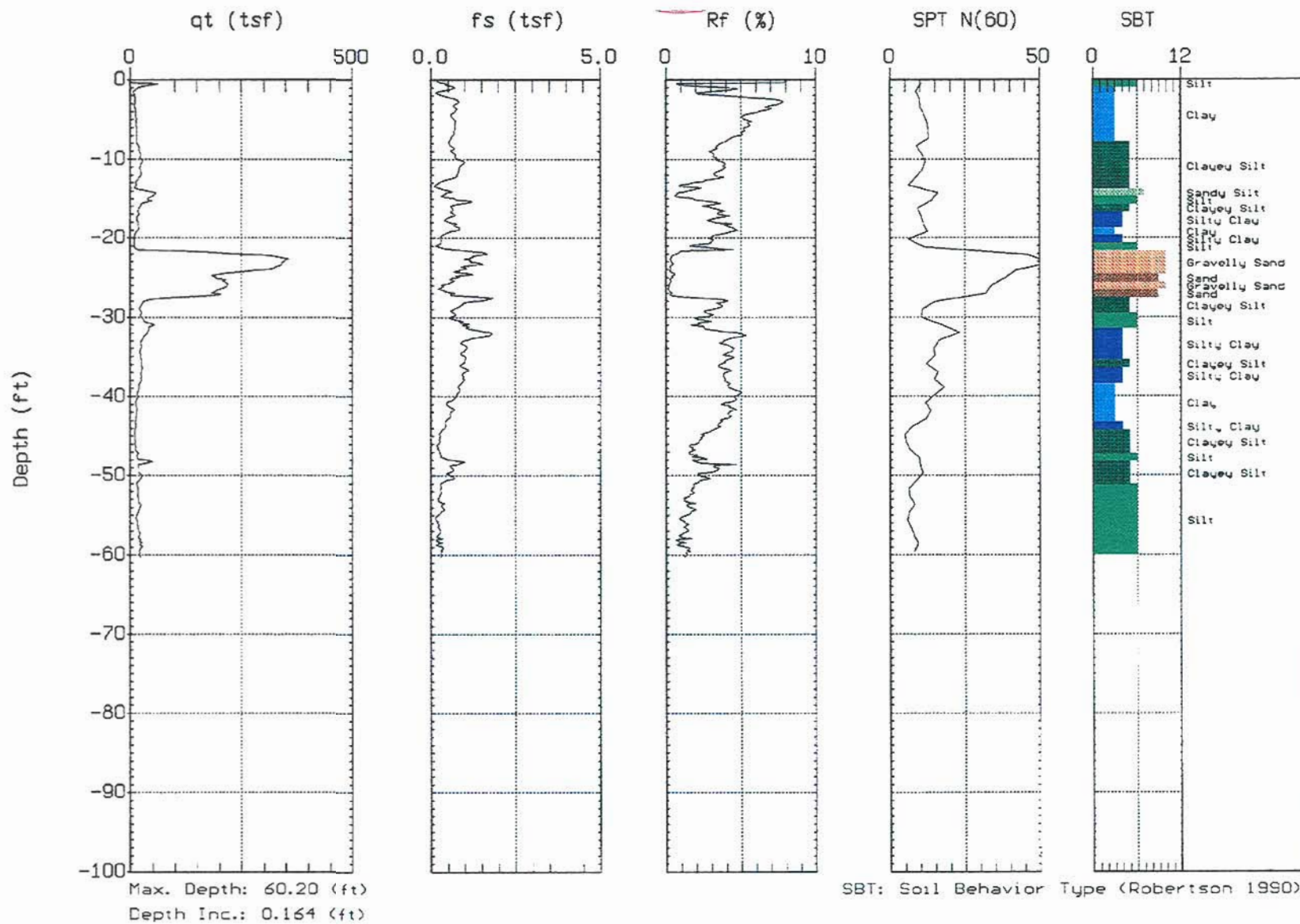
6/05*EB

CONE PENETRATION TEST - CPT-3



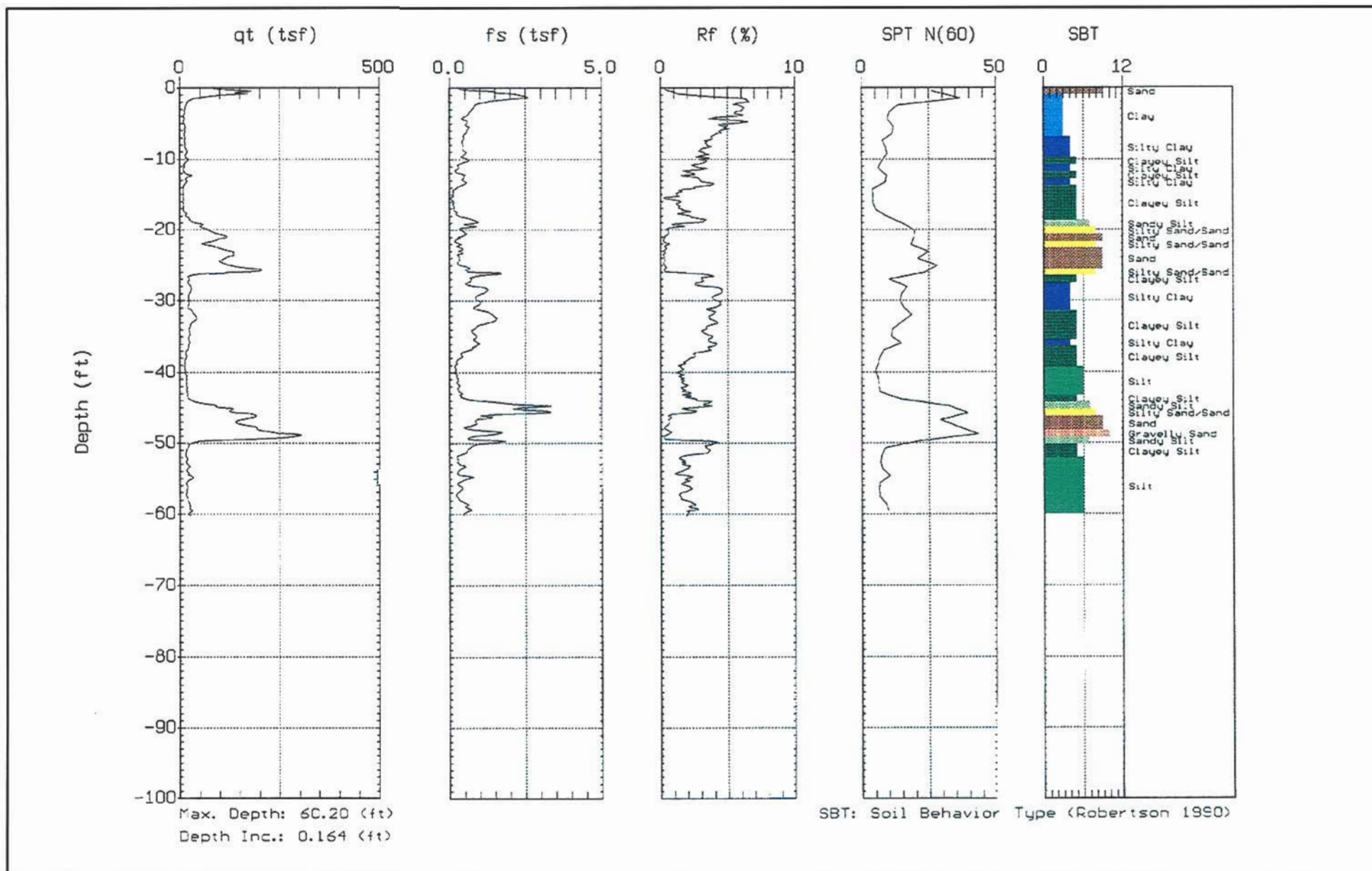
6/05*EB

CONE PENETRATION TEST - CPT-4



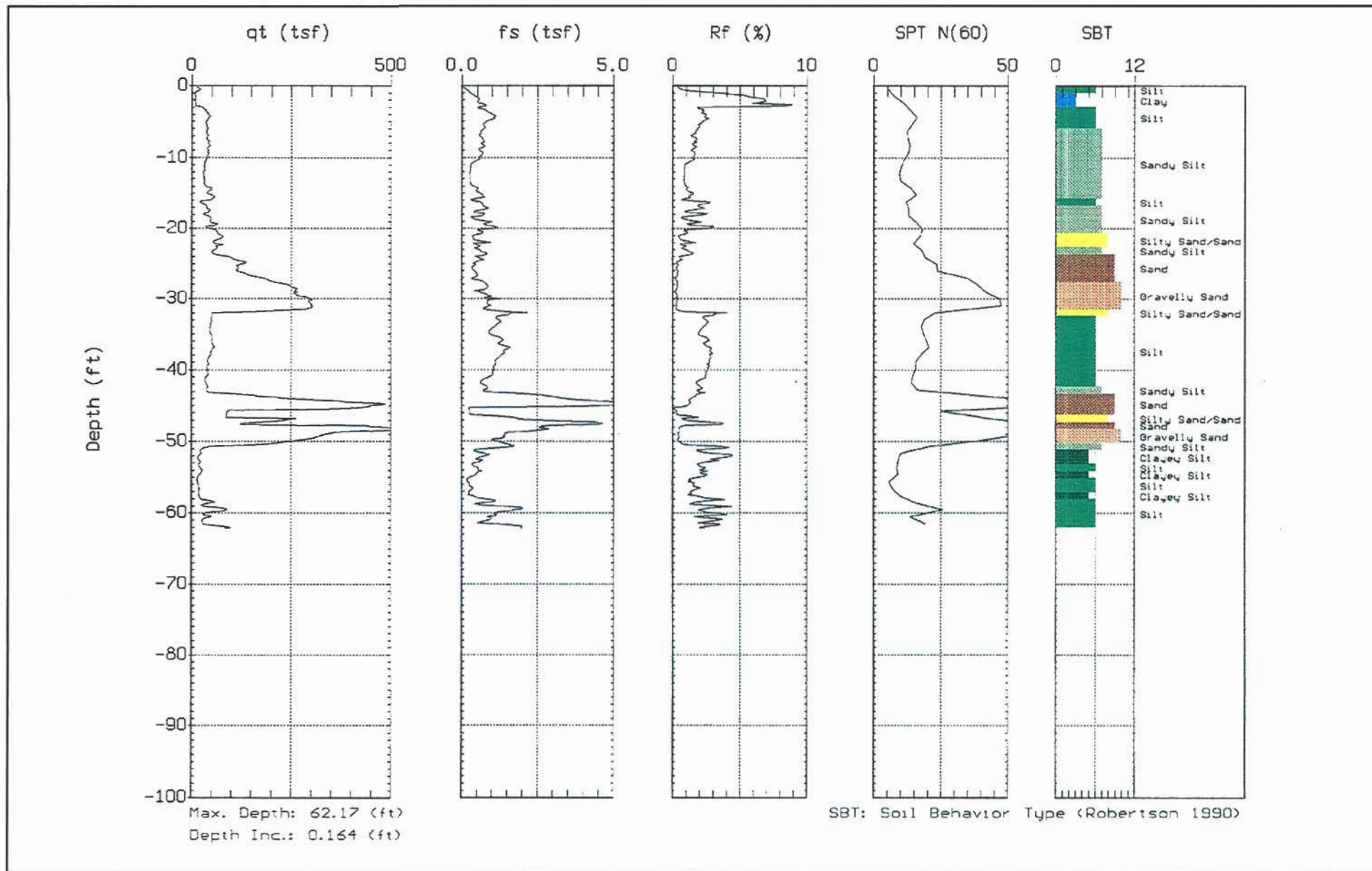
6/05*EB

CONE PENETRATION TEST - CPT-5



6/05'EB

CONE PENETRATION TEST - CPT-6



6/05'EB

CONE PENETRATION TEST - CPT-7

APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 34 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 25 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

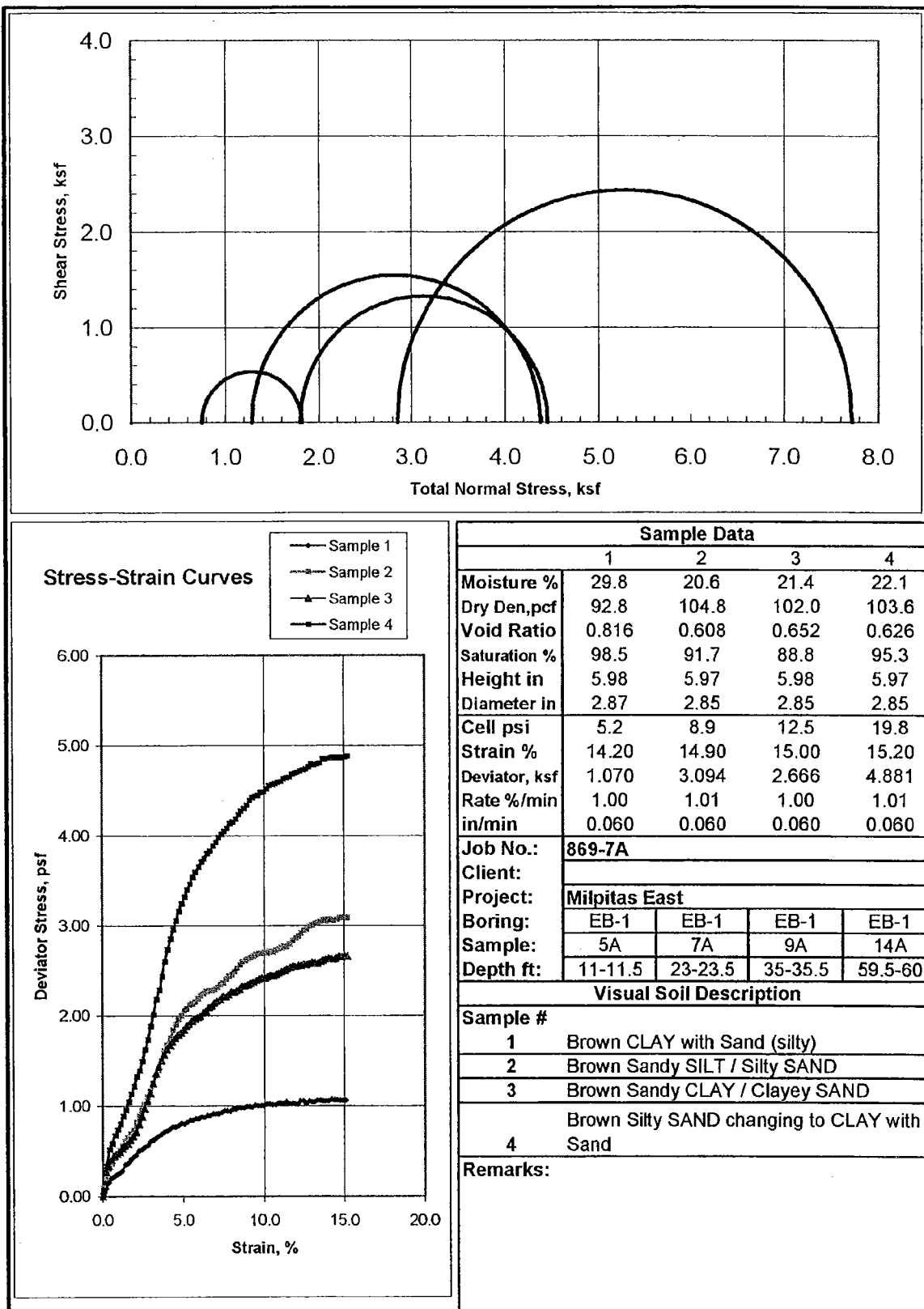
Plasticity Index: Plasticity Index (PI) tests (ASTM D4318) were performed on two samples of the subsurface soils to measure the range of water contents over which these materials exhibit plasticity. The PI was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on Figure B-1 and on the logs of the borings at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was performed on 10 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Unconsolidated-Undrained Triaxial Shear Test: Unconsolidated-Undrained Triaxial Shear tests were performed on four samples of the subsurface clayey soil samples to find the undrained shear strength of these soils. Results of these tests are shown on the boring logs at the appropriate depths and presented graphically on Figure B-2.

Consolidation: One consolidation test (ASTM D2435) was performed on an undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation test are presented graphically on Figure B-3.

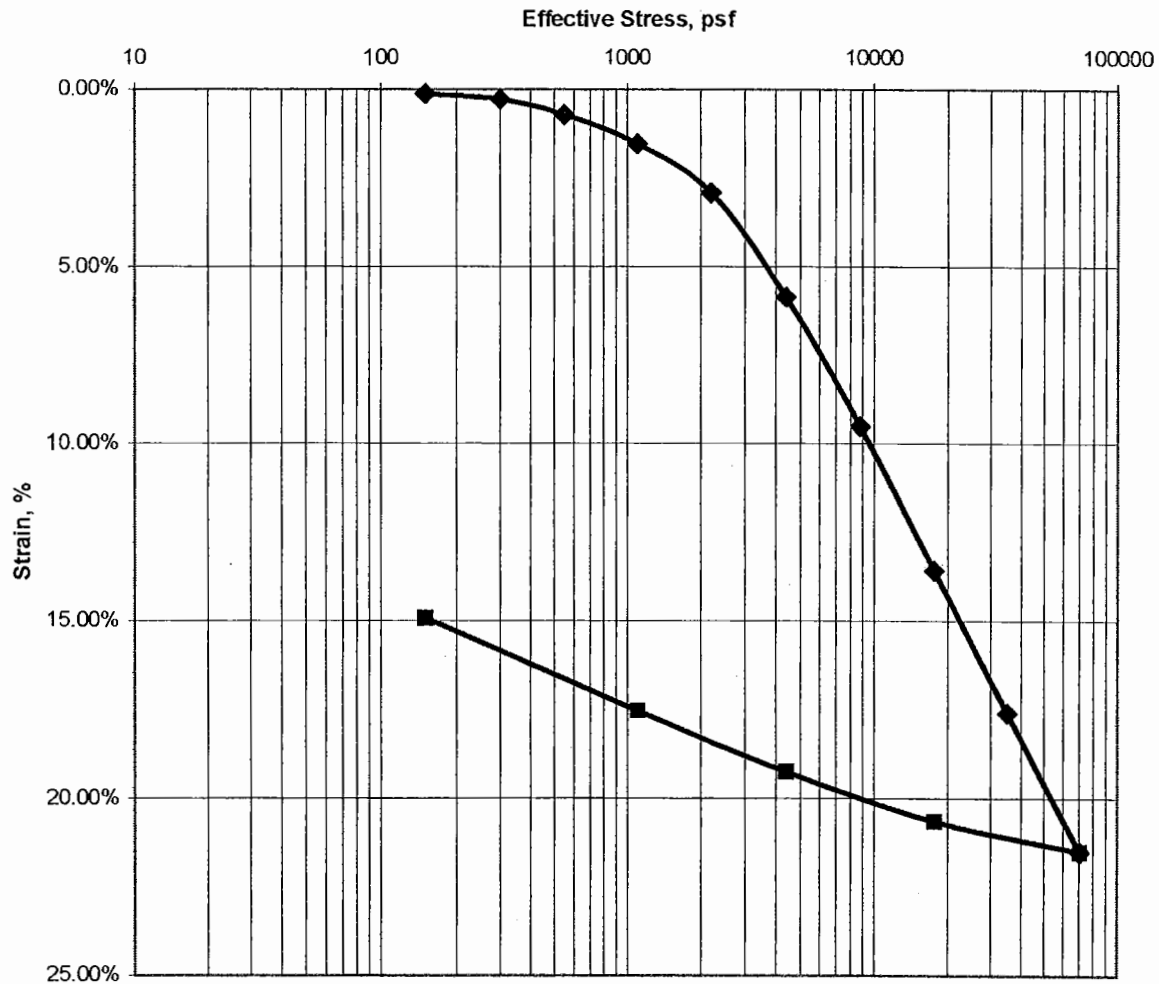
* * * * *



TRIAXIAL UU TEST

Job No.:	869-7A	Boring:	EB-1	Run By:	MD
Client:		Sample:	5A	Reduced:	MJ
Project:	Midtown East Parking Structure	Depth, ft.:	11.5	Checked:	PJ
Soil Type:	Brown Lean Clay, some Sand			Date:	6/10/2005

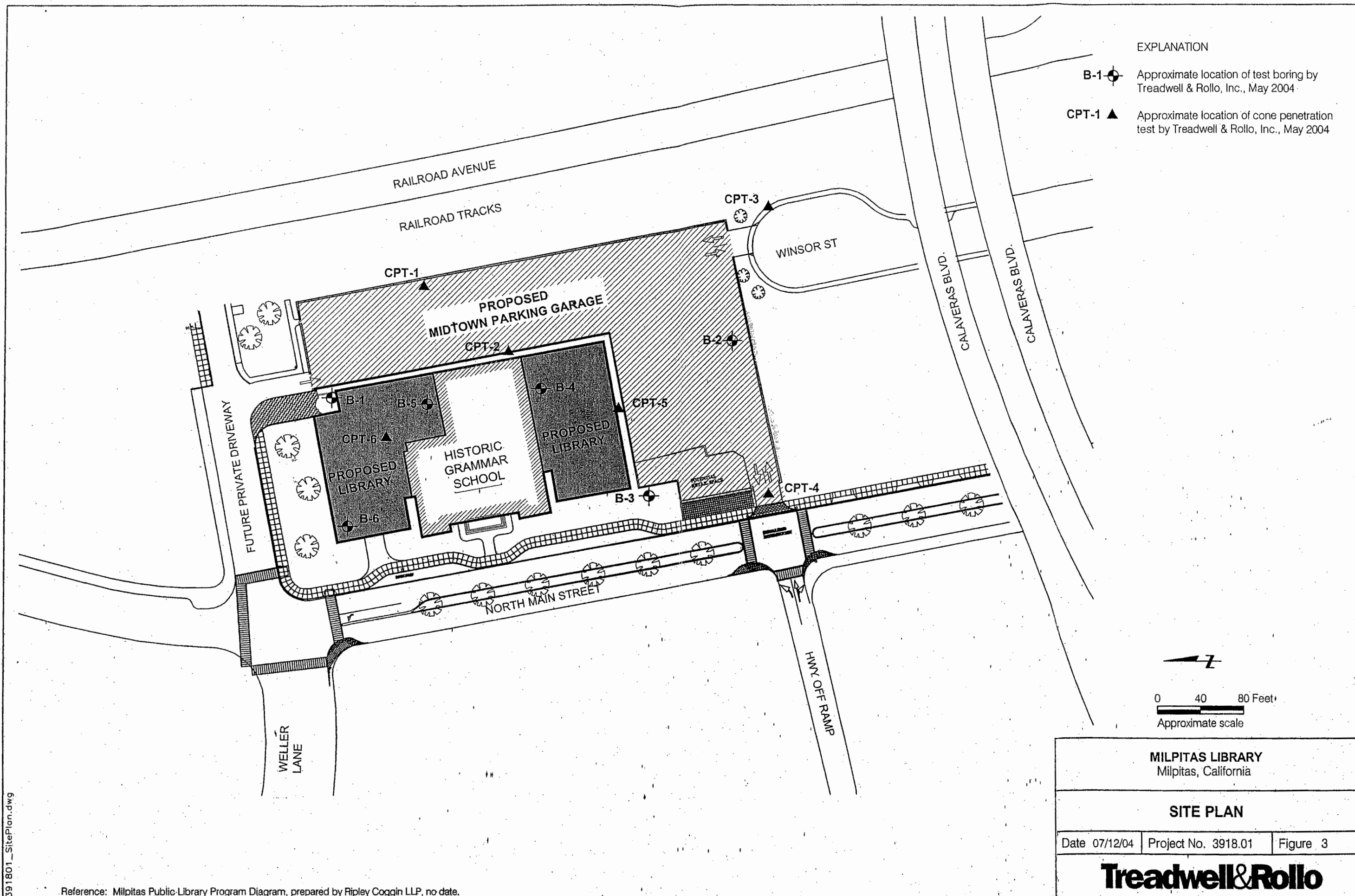
Strain-Log-P Curve



Ass. Gs = 2.7	Initial	Final	Remarks:
Moisture %:	27.7	20.3	
Density, pcf:	95.9	109.0	
Void Ratio:	0.758	0.547	
% Saturation:	98.7	100	

CONSOLIDATION TEST

APPENDIX C
PREVIOUS FIELD INVESTIGATION AND LABORATORY TEST DATA BY OTHERS



391801_SitePlan.dwg

Reference: Milpitas Public Library Program Diagram, prepared by Ripley Coggin LLP, no date.

APPENDIX A
Logs of Test Borings and Cone Penetration Test Results

PROJECT:

MILPITAS LIBRARY
Milpitas, California

Log of Boring B-2

PAGE 1 OF 4

Boring location: See Site Plan, Figure 3

Logged by: C. Divis

Date started: 5/13/04

Date finished: 5/13/04

Drilling method: Rotary wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Safety

LABORATORY TEST DATA

Sampler: Sprague & Henwood (S&H), Shelby Tube (ST), Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	SPT N-Value								
1					4-inch-thick layer of asphalt-concrete over 6-inch-thick layer of aggregate base						
2					CLAY (CH)						
3	S&H		10	CH	dark gray, stiff, moist, trace of organics and sand Corrosivity Test, see Appendix B LL = 64, PI = 44, see Appendix B					27.6	94
4											
5	S&H		9								
6											
7					CLAY (CL)						
8				CL	olive-gray, medium stiff, wet						
9					Unconsolidated Undrained Triaxial Test, see Appendix B	TxUU	600	1,380		20.9	107
10	ST				Consolidation Test, see Appendix B					21.0	105
11				CL	CLAY with SAND (CL)						
12					olive-brown, stiff, wet, fine sand	PP		1,250			
13											
14					SAND with SILT (SP-SM)						
15	S&H		14	SP-SM	brown, medium dense, wet, fine to medium sand				9.8		
16											
17					CLAYEY SAND (SC)						
18				SC	olive-brown, medium dense, wet, fine sand						
19											
20	S&H		10	CL	SANDY CLAY (CL)						
21					brown, stiff, wet						
22					SILTY SAND (SM)						
23				SM	gray, medium dense, wet, fine sand						
24											
25	SPT		24		SAND with GRAVEL and SILT (SP-SM)				13.1		
26				SP-SM	gray, medium dense, wet, medium to coarse sand						
27											
28					SAND with GRAVEL (SP)						
29	SPT		24	SP	gray, medium dense, wet, medium to coarse sand, fine to coarse gravel						
30											

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3918.01

Figure:

A-2a

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

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Log of Boring B-2

PAGE 2 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content %	Dry Density Lbs/Cu Ft
31	SPT		24		CLAY (CL) olive, very stiff, wet						
32											
33											
34											
35	S&H		17	CL							
36											
37											
38											
39					clayey sand lens at 39 to 39.5 feet						
40	SPT		15		SANDY CLAY (CL) olive-brown, stiff to very stiff, wet						
41				CL							
42											
43											
44					CLAYEY SAND with GRAVEL (SC) mottled brown and gray, dense, hard, wet						
45	S&H		36	SC							
46											
47											
48					CLAY (CL) mottled gray and olive, very stiff, wet occasional gravel						
49											
50											
51	S&H		18								
52											
53				CL							
54											
55											
56											
57											
58											
59	S&H		16		fine sand						
60											

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

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Figure:

A-2b

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Log of Boring B-2

PAGE 3 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H		16		CLAY (CL) (continued)						
62											
63											
64											
65											
66											
67											
68											
69											
70	S&H		26		increase in sand content, olive						
71											
72											
73											
74											
75				CL							
76											
77											
78											
79											
80	S&H		30								
81											
82											
83											
84											
85											
86											
87											
88											
89	S&H		28								
90											

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Figure:

A-2c

PROJECT:

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Log of Boring B-2

PAGE 4 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		28	CL	CLAY (CL) (continued)						
92											
93											
94											
95											
96											
97											
98											
99					mottled olive and gray						
100	S&H		25								
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 100.5 feet.
Boring backfilled with cement grout.
Groundwater obscured by drilling method.

¹ S&H blow counts converted to SPT N-values using a factor of 0.6.

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Figure:

A-2d

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

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Log of Boring B-3

PAGE 1 OF 4

Boring location: See Site Plan, Figure 3

Logged by: C. Davis

Date started: 5/12/04

Date finished: 5/12/04

Drilling method: Rotary wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Shelby Tube (ST), Standard Penetration Test (SPT)

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	SPT N-Value ¹								
1				SC	1-inch-thick layer of asphalt-concrete over 6-inch-thick layer of aggregate base						
2					CLAYEY SAND (SC) brown, loose to medium dense, dry to moist R-value = 26, see Appendix B						
3	S&H		14		CLAY (CH) mottled dark gray and brown, stiff to very stiff, moist, trace of organics						
4											
5	S&H		12	CH							
6											
7					▽ (5/13/04)						
8											
9					SANDY CLAY (CL) olive-brown, stiff, moist to wet						
10	ST		300 psi	CL							
11					very stiff						
12											
13											
14					CLAYEY SAND (SC) olive-brown, medium dense, wet, fine sand						
15	SPT		19	SC							
16											
17				CL	CLAY (CL) dark gray, medium stiff to stiff, wet						
18											
19				CL	SANDY CLAY (CL) olive-brown, stiff, wet, fine sand						
20	S&H		12								
21					SAND with GRAVEL and SILT (SP-SM) brown/black, medium dense, wet						
22				SP- SM							
23											
24											
25	SPT		18								
26					CLAY (CL) olive-brown, stiff, wet						
27				CL							
28											
29	S&H		16		trace of sand						
30											

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3918.01

Figure:

A-3a

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

PROJECT:

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Log of Boring B-3

PAGE 2 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		16		CLAY (CL) (continued)						
32											
33											
34				CL	Unconsolidated Undrained Triaxial Test, see Appendix B	TxUU	1,600	1,200		24.5	101
35					Consolidation Test, see Appendix B						
36	ST				stiff to very stiff					24.5	99
37						PP		2,000			
38					CLAY with SAND (CL) olive-brown, stiff to very stiff, wet, fine sand						
39											
40	S&H		16	CL							
41											
42											
43					CLAY (CL) mottled tan and olive-brown, very stiff to hard, wet						
44											
45	S&H		37								
46											
47											
48											
49				CL	very stiff						
50	S&H		16								
51											
52											
53											
54											
55											
56					SILT with SAND (ML) olive-brown, stiff, wet, fine sand						
57											
58				ML							
59											
60	S&H		13		Partial Size Distribution, see Appendix B				76.6		

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

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Figure:

A-3b

PROJECT:

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Log of Boring B-3

PAGE 3 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H		13	ML	SILT with SAND (ML) (continued) LL = 27, PI = 4, see Appendix B						
62											
63											
64											
65					interbedded sand and clay						
66				SM							
67					SILTY SAND (SM) olive, medium dense, wet, fine sand						
68					Partial Size Distribution, see Appendix B PI = Non-Plastic, see Appendix B				48.4		
69	S&H		24								
70											
71				SM							
72											
73					interbedded sand and silt						
74											
75											
76				SM							
77											
78											
79					SILTY SAND (SM) gray, dense, wet, fine to medium sand						
80	S&H		32		CLAY (CL) olive-gray, hard, wet						
81				CL							
82											
83											
84											
85											
86				CL							
87											
88											
89	S&H		19		olive-brown, very stiff						
90											

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3918.01

Figure:

A-3c

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

PROJECT:

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Log of Boring B-3

PAGE 4 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		19		CLAY (CL) (continues)						
92											
93											
94											
95				CL							
96											
97											
98											
99											
100	S&H		28								
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 100.5 feet.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 6.5 feet.

¹ S&H blow counts converted to SPT N-values using a factor of 0.6.

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Figure:

A-3d

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

PROJECT:

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Log of Boring B-4

PAGE 1 OF 4

Boring location: See Site Plan, Figure 3

Logged by: C. Davis

Date started: 5/10/04

Date finished: 5/10/04

Drilling method: Rotary wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Shelby Tube (ST)

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	SPT N-Value ¹								
1					4-inch-thick layer of gravel						
2					CLAY (CH)						
3	S&H		13		dark gray, stiff, moist					23.7	100
4					LL = 53, PI = 34, see Appendix B						
5	S&H		10	CH							
6					▽ (5/11/04)						
7											
8											
9											
10	S&H		17		CLAY with SAND (CL)						
11					olive-brown, very stiff, wet, fine sand, slight petroleum						
12				CL	hydrocarbon odor						
13											
14					CLAY (CL)	TxUU	800	1,040		28.6	96
15	ST		100 to 300 psi		olive-brown, stiff, wet					30.6	90
16					Unconsolidated Undrained Triaxial Test, see						
17				CL	Appendix B	PP		1,750			
18											
19											
20	ST				sand lens at 20 to 21 feet						
21					CLAYEY SAND (SC)						
22	S&H		8	SC	olive-brown, loose, wet						
23											
24					CLAY (CL)						
25	S&H		23		mottled olive-brown and red, very stiff, wet, with						
26				CL	variable sand content, fine sand						
27											
28					CLAY with SAND (CL)						
29	S&H		17	CL	olive-brown, very stiff, wet, fine sand						
30											

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Figure:

A-4a

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Log of Boring B-4

PAGE 2 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H		17	CL	SANDY CLAY (CL) (continued)						
32					SANDY CLAY (CL) olive-brown, very stiff, wet, fine sand, occasional gravel						
33											
34											
35	S&H		16								
36				CL							
37											
38											
39					stiff						
40	S&H		13								
41											
42											
43					CLAY (CL) olive-brown, very stiff, wet						
44											
45	S&H		28	CL							
46											
47											
48					SANDY CLAY (CL) olive-brown, stiff, wet, fine sand						
49											
50	S&H		13								
51											
52											
53				CL							
54											
55											
56											
57											
58											
59	S&H		14								
60											

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

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3918.01

Figure:

A-4b

PROJECT:

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Log of Boring B-4

PAGE 3 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H		14	CL	SANDY CLAY (CL) (continued)						
62											
63											
64											
65				CL	CLAY (CL) olive-brown, very stiff, wet						
66											
67											
68											
69				CL	increase in sand content at 70 feet						
70	S&H		26								
71											
72											
73				CL							
74											
75											
76											
77				CL							
78											
79											
80	S&H		29								
81				CL							
82											
83											
84											
85				CL							
86											
87											
88											
89	S&H		22	CL							
90											

TEST GEOTECH LOG 391801.GPJ TR.GDT 7/14/04

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3918.01

Figure:

A-4c

PROJECT:

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Log of Boring B-4

PAGE 4 OF 4

DEPTH (feet)	SAMPLES			LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	S&H		22		CLAY (CL) (continued)						
92											
93											
94											
95				CL							
96											
97											
98											
99					hard						
100	S&H		31								
101											
102											
103											
104											
105											
106											
107											
108											
109											
110											
111											
112											
113											
114											
115											
116											
117											
118											
119											
120											

Boring terminated at a depth of 100.5 feet.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 6.5 feet.

¹ S&H blow counts converted to SPT N-values using a factor of 0.6.

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Figure:

A-4d









UNIFIED SOIL CLASSIFICATION SYSTEM			
Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

SAMPLE DESIGNATIONS/SYMBOLS

GRAIN SIZE CHART		
Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4	76.2 to 4.76
	3" to 3/4" 3/4" to No. 4	76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200	4.76 to 0.074
	No. 4 to No. 10	4.76 to 2.00
	No. 10 to No. 40 No. 40 to No. 200	2.00 to 0.420 0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

▽ Unstabilized groundwater level

▽ Stabilized groundwater level

	Sample taken with split-barrel sampler other than Standard Penetration Test sampler. Darkened area indicates soil recovered
	Classification sample taken with Standard Penetration Test sampler
	Undisturbed sample taken with thin-walled tube
	Disturbed sample
	Sampling attempted with no recovery
	Core sample
	Analytical laboratory sample
	Sample taken with Direct Push sampler

SAMPLER TYPE

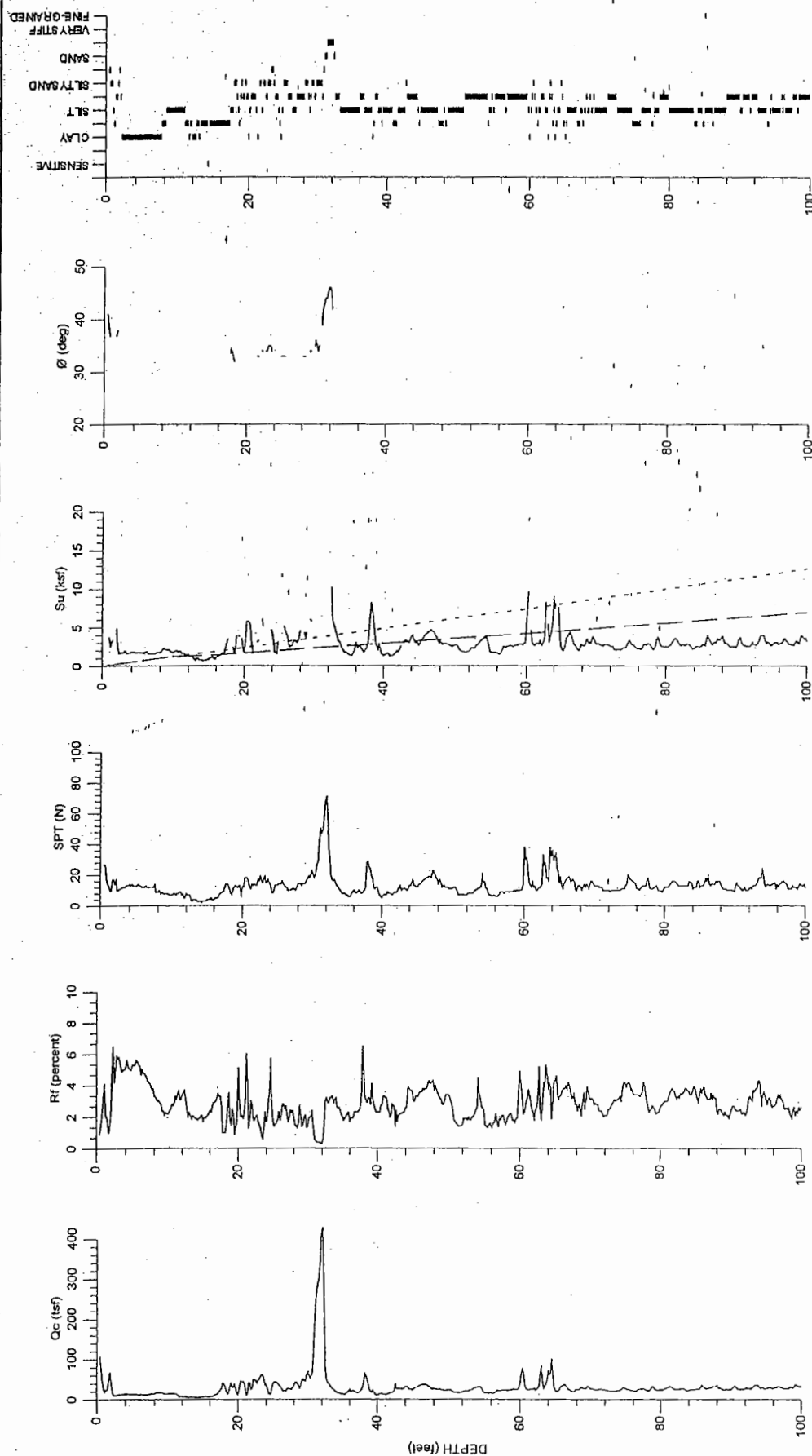
C	Core barrel	PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter	S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube	SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube	ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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CLASSIFICATION CHART

Treadwell&Rollo

Date 05/25/04 Project No. 3918.01 Figure A-7



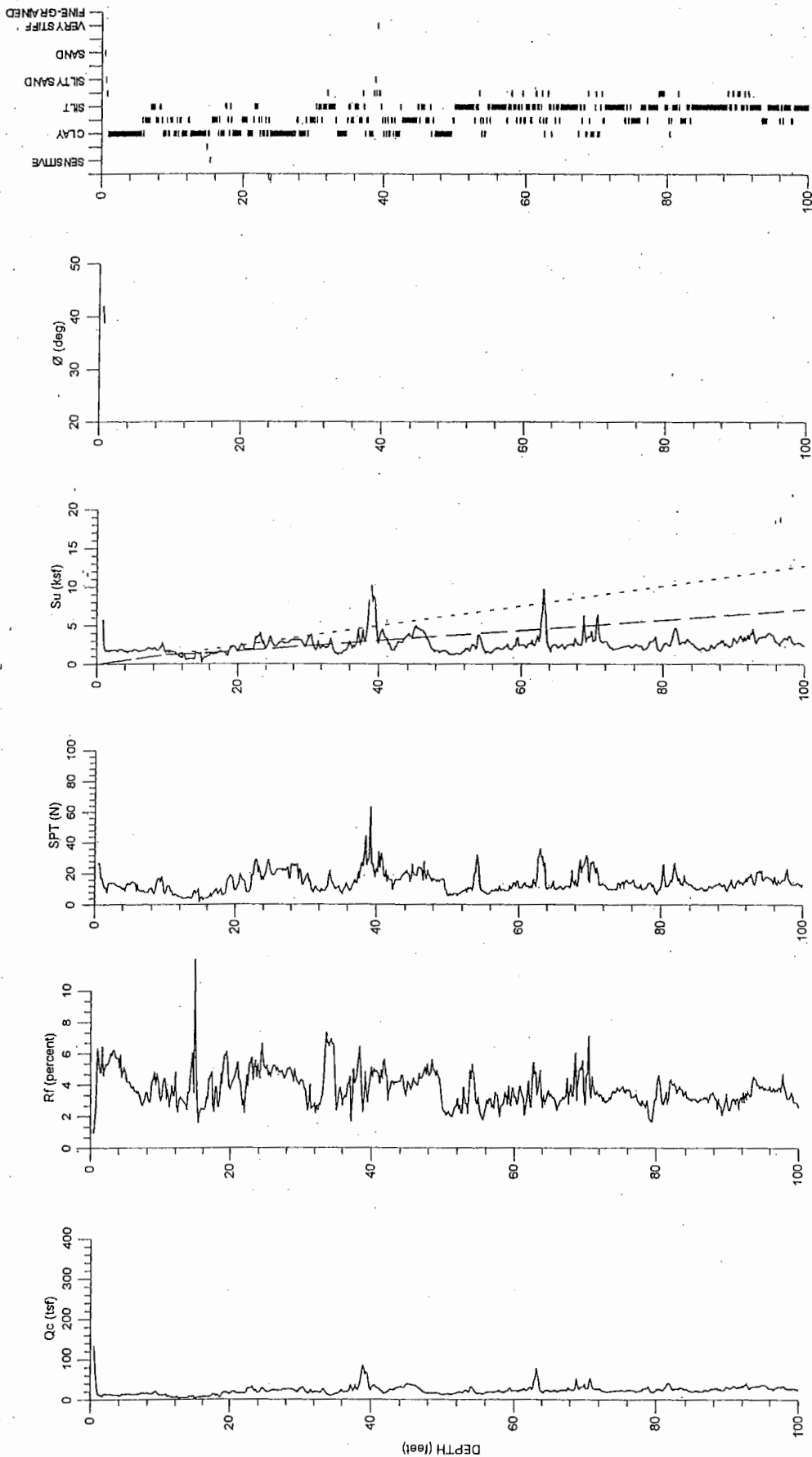
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CONE PENETRATION TEST RESULTS
CPT-1

Date 05/27/04 Project No. 3918.01 Figure A-8

Treadwell & Rollo

Terminated at 100 feet
Groundwater was measured at 9 feet.
Date performed: 05/25/04.



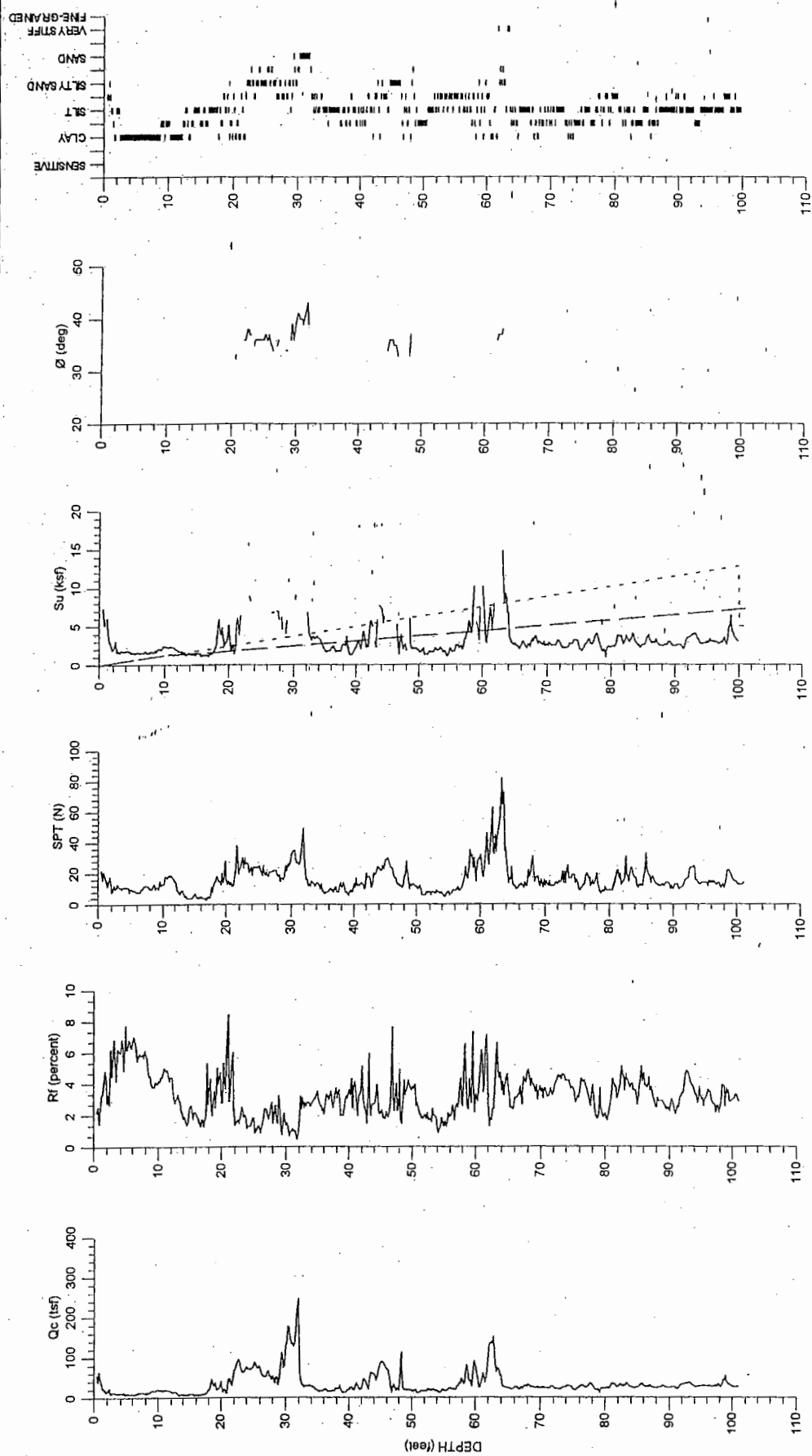
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CONE PENETRATION TEST RESULTS CPT-2

Date 05/14/04 Project No. 3918.01 Figure A-9

Treadwell & Rollo

Terminated at 100 feet
Groundwater was measured at 9.2 feet.
Date performed: 05/13/04.



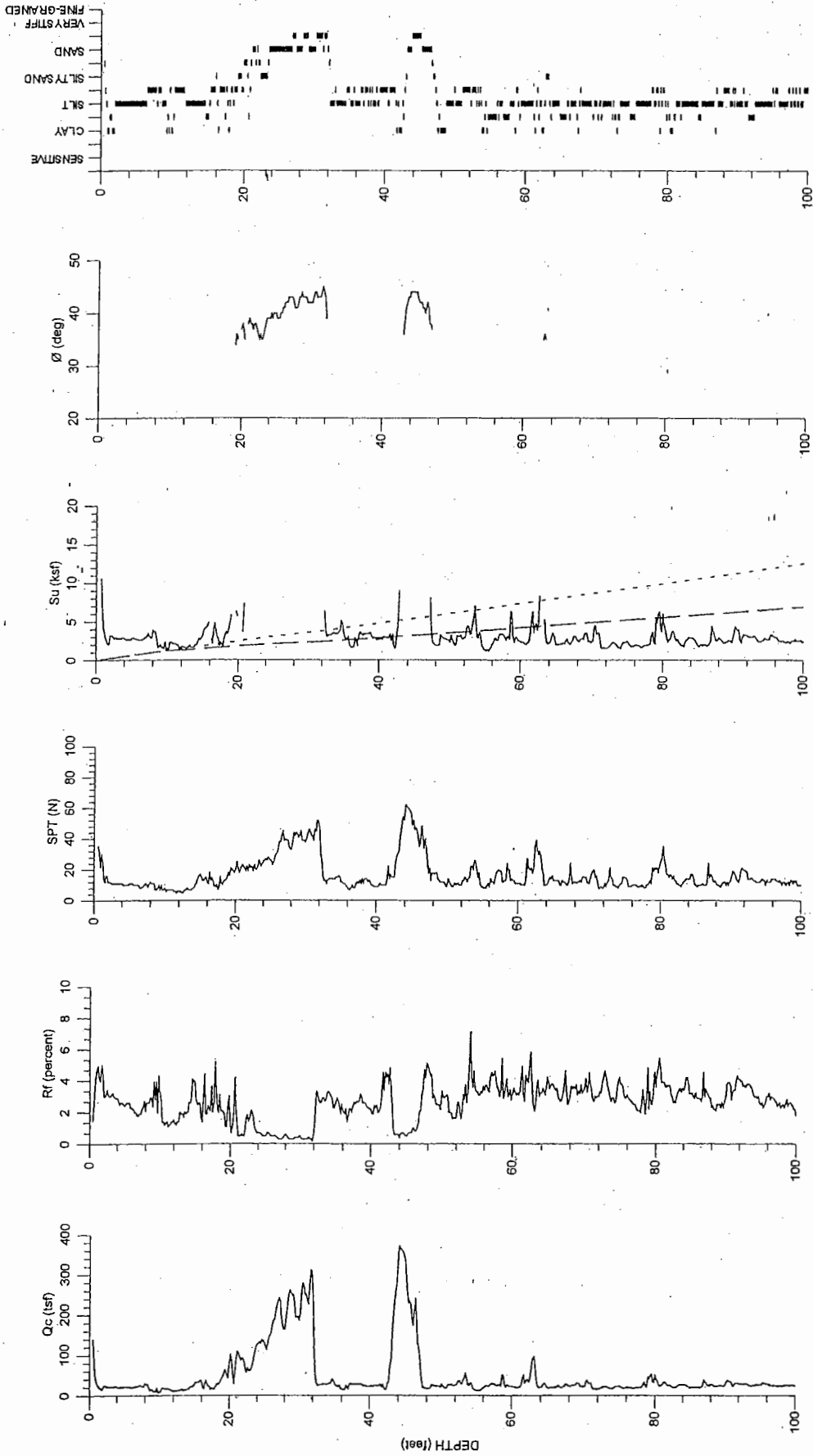
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CONE PENETRATION TEST RESULTS CPT-3

Date 05/14/04 Project No. 3918.01 Figure A-10

Treadwell & Rollo

Terminated at 100 feet
Groundwater was measured at 7 feet.
Date performed: 05/13/04.



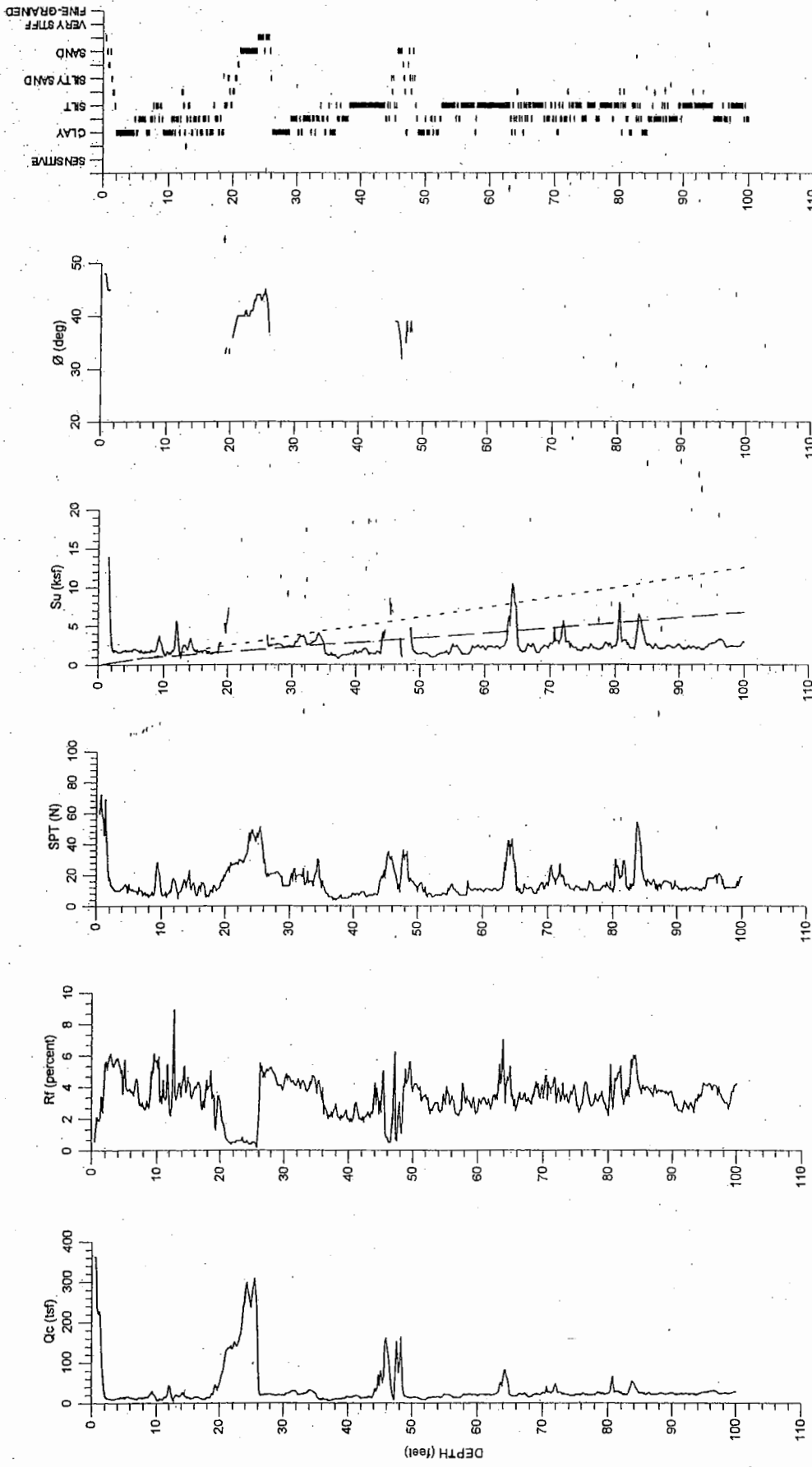
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CONE PENETRATION TEST RESULTS CPT-4

Date 05/27/04 Project No. 3918.01 Figure A-11

Treadwell & Rollo

Terminated at 100 feet
Groundwater was measured at 9.7 feet.
Date performed: 05/25/04.



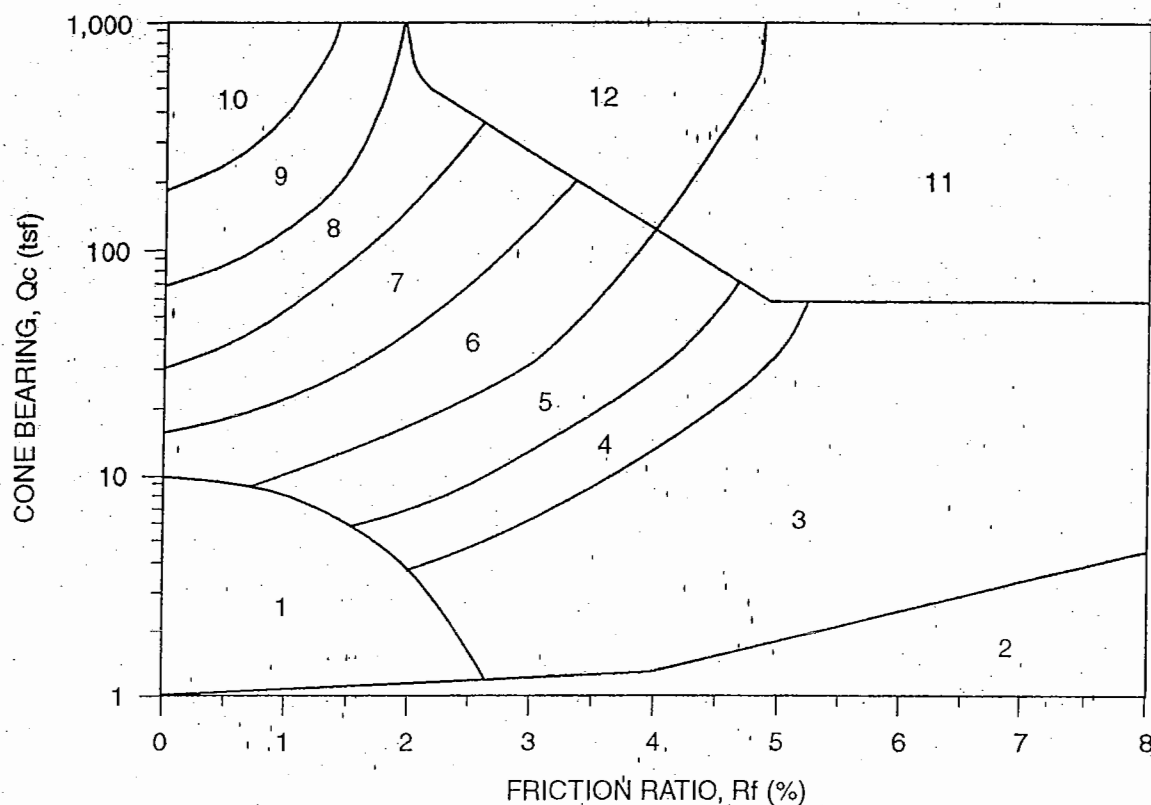
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CONE PENETRATION TEST RESULTS CPT-5

Date 05/14/04 Project No. 3918.01 Figure A-12

Treadwell & Rollo

Terminated at 100 feet
Groundwater was measured at 7 feet.
Date performed: 05/13/04.



ZONE	Q_c/N^1	S_u Factor $(N_k)^2$	SOIL BEHAVIOR TYPE ¹
1	2	15 (10 for $Q_c \leq 9$ tsf)	Sensitive Fine-Grained
2	1	15 (10 for $Q_c \leq 9$ tsf)	Organic Material
3	1	15 (10 for $Q_c \leq 9$ tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented

Q_c = Tip Bearing

F_s = Sleeve Friction

$R_f = F_s/Q_c \times 100$ = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. Robertson, 1986, Olsen, 1988.

2. Bonaparte & Mitchell, 1979 (young Bay Mud $Q_c \leq 9$).

Estimated from local experience (fine-grained soils $Q_c > 9$).

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CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Treadwell&Rollo

Date 07/07/04

Project No. 3918.01

Figure A-14

APPENDIX B Laboratory Test Results



Moisture-Density-Porosity Report

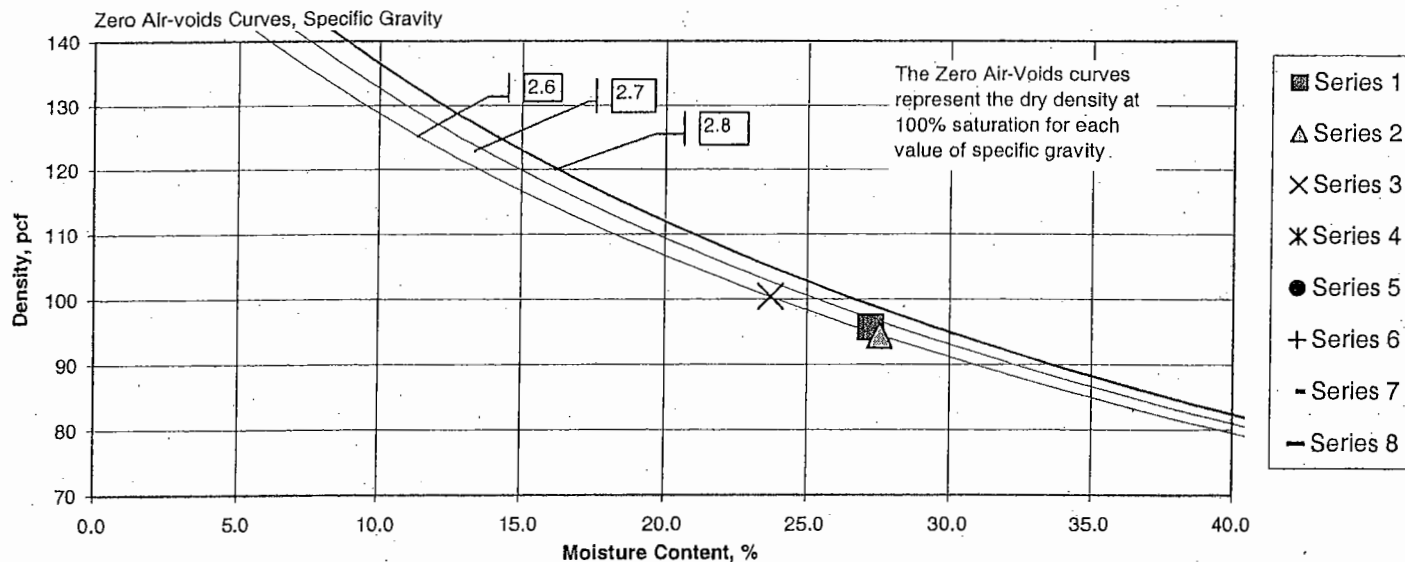
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 Project: Milpitas Library - 3918.01

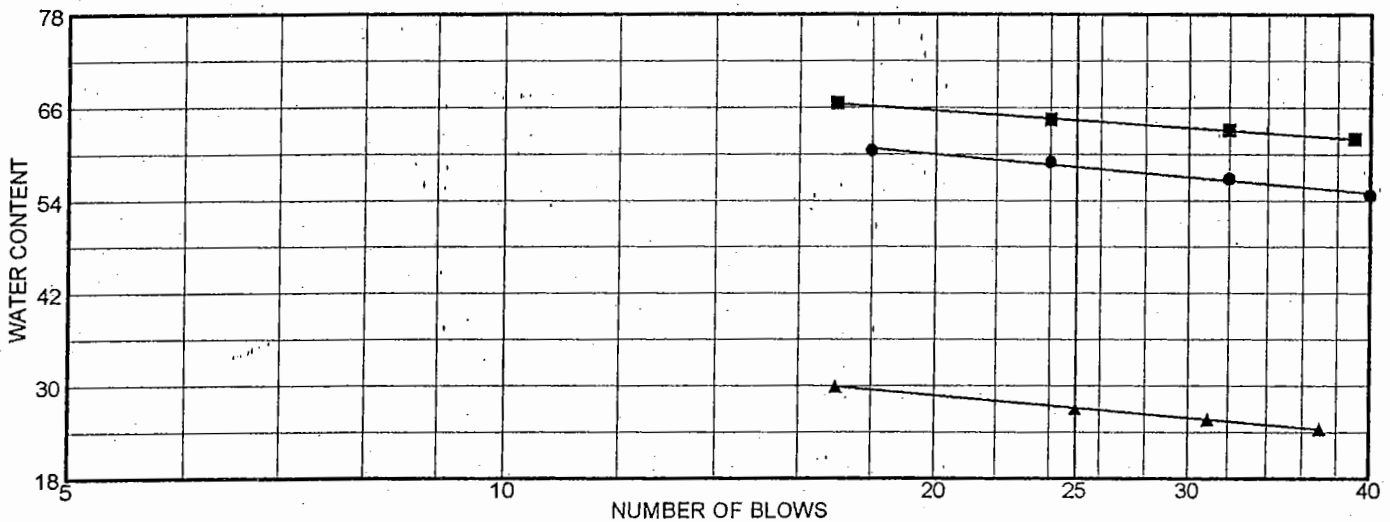
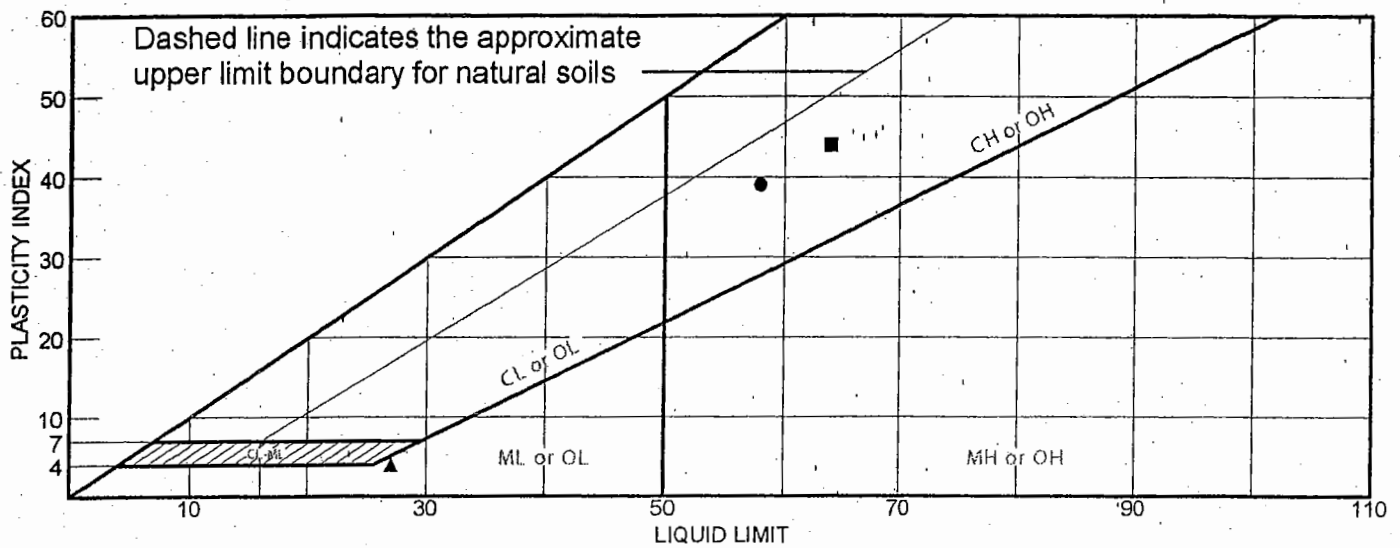
Date: 06/04/04
 By: MJ
 Remarks:

Boring:	B-1	B-2	B-4					
Sample:	1	1	1					
Depth, ft:	3	3	3					
Visual Description:	Dark Gray Fat CLAY	Dark Gray Fat CLAY	Dark Gray Fat CLAY					
Actual G_s								
Assumed G_s	2.70	2.70	2.70					
Total Vol cc	146.90	146.90	146.90					
Vol Solids, cc	83.26	82.19	87.41					
Vol Voids, cc	63.64	64.71	59.49					
Moisture, %	27.3	27.6	23.7					
Wet Unit wt, pcf	121.7	120.4	124.2					
Dry Unit wt, pcf	95.6	94.4	100.4					
Saturation, %	96.3	94.5	94.1					
Porosity, %	43.3	44.0	40.5					
Void Ratio	0.764	0.787	0.681					
Series	1	2	3	4	5	6	7	8

Moisture-Density



LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark gray Fat CLAY	58	19	39			
■	Dark gray Fat CLAY	64	20	44			
▲	Olive-brown SILT with SAND	27	23	4	100.0	76.6	ML
◆	Olive Silty SAND		24	NP	99.8	48.4	SM

Project No. 010-831 Client: Treadwell & Rollo

Project: Milpitas Library - 3918.01

● Source: B-1 Sample No.: 1 Elev./Depth: 3'
 ■ Source: B-2 Sample No.: 1 Elev./Depth: 3'
 ▲ Source: B-3 Sample No.: 14 Elev./Depth: 59'
 ◆ Source: B-3 Sample No.: 15 Elev./Depth: 70'

Remarks:

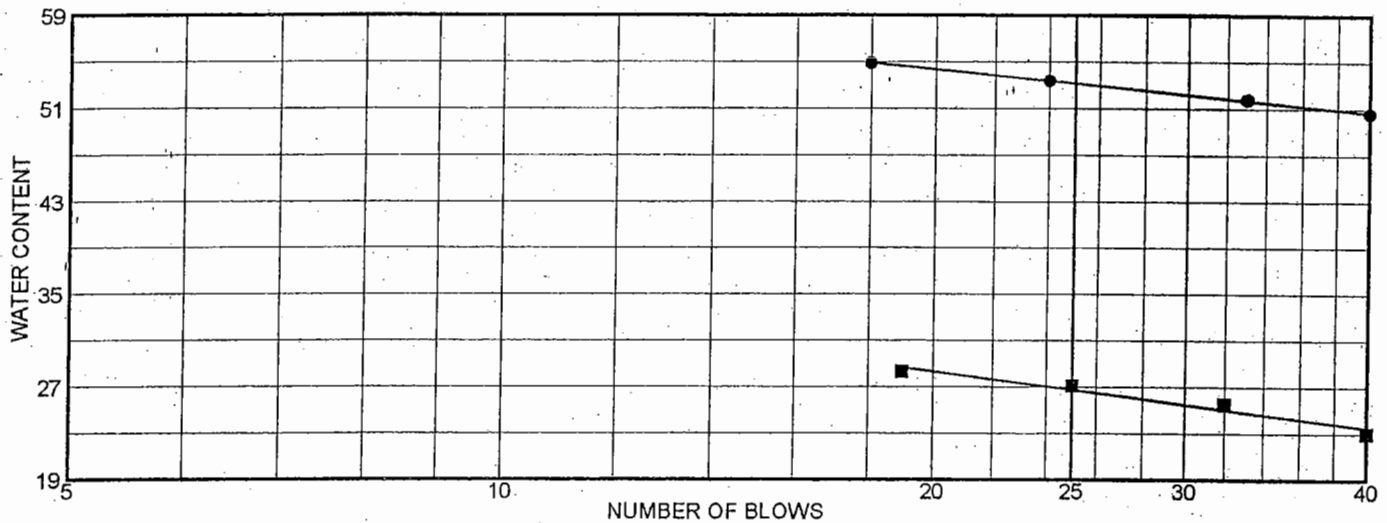
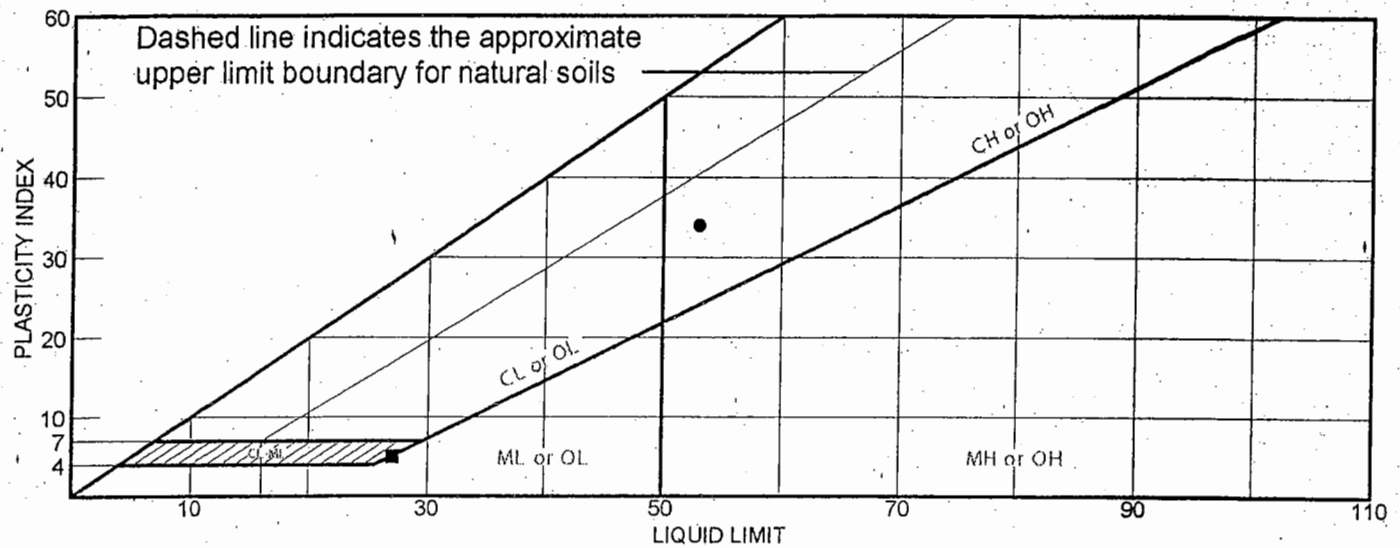
◆ Non-plastic: could not roll out and sample slides in the bowl.

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	% < #40	% < #200	USCS
●	Dark gray Fat CLAY	53	19	34			
■	Olive-gray Silty CLAY	27	22	5			

Project No. 010-831 Client: Treadwell & Rollo

Project: Milpitas Library - 3918.01

● Source: B-4

Sample No.: 1

Elev./Depth: 3'

■ Source: B-6

Sample No.: 14

Elev./Depth: 60'

Remarks:

●
■

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure



#200 Sieve Wash Analysis ASTM D 1140

Job No.: 010-831

Client: Treadwell and Rollo

Project: Milpitas Library

Project No.: 3918.01

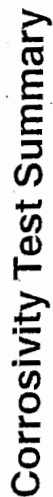
Date: 6/4/2004

Run By: MD

Checked By: DC

Boring:	B-1	B-1	B-2	B-2	B-2	B-5	B-6	
Sample:	8	11	5	8	24	6	5	
Depth, ft.:	19	29	15	24	24	24	19	
Soil Type:	Olive-gray SAND with Silt	Olive-gray SAND with Gravel	Brown SAND with Silt	Gray Silty SAND	Brown Silty SAND	Brown Silty SAND	Brown Silty SAND	
Wt of Dish & Dry Soil, gm	545.8	553.3	436.5	404.0	517.3	556.8		
Weight of Dish, gm	179.4	121.5	180.5	156.2	165.0	178.5		
Weight of Dry Soil, gm	366.4	431.8	256.0	247.8	352.3	378.3		
Wt. Ret. on #4 Sieve, gm	1.6	111.2	0.0	2.1	1.4	0.0		
Wt. Ret. on #200 Sieve, gm	335.6	414.2	230.8	215.4	200.2	253.0		
% Gravel	0.4	25.8	0.0	0.8	0.4	0.0		
% Sand	91.2	70.2	90.2	86.1	56.4	66.9		
% Silt & Clay	8.4	4.1	9.8	13.1	43.2	33.1		

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine



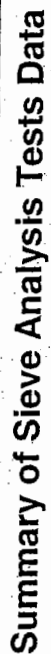
Checked: DI

1

Doc: No. 004004

Doc: No. 004004

[illegible]

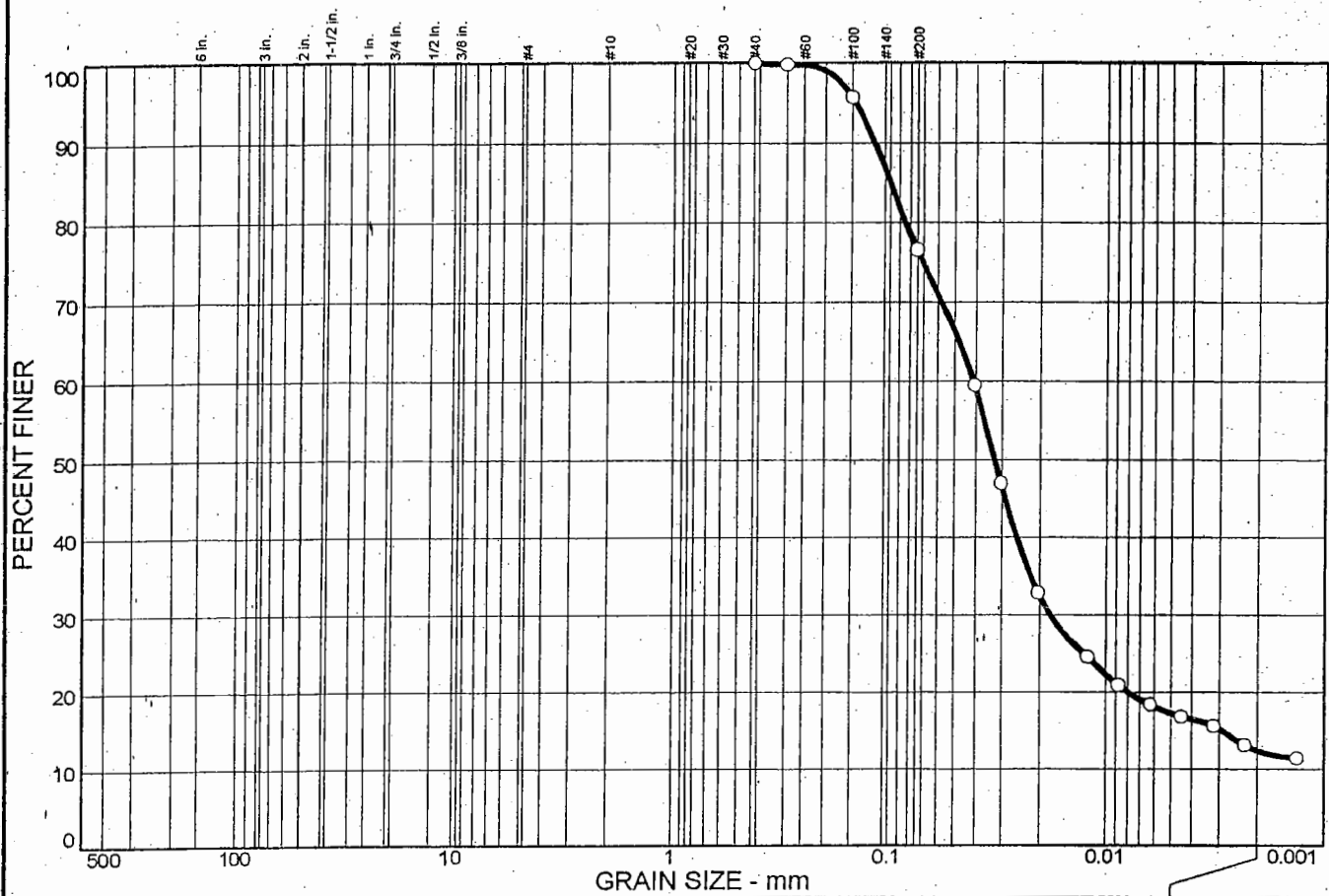


Summary of Sieve Analysis Tests Data

Remarks:

[illegible]

PARTICLE SIZE DISTRIBUTION TEST REPORT



% +3"	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.0	23.4	64.2	12.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#40	100.0		
#50	99.8		
#100	95.7		
#200	76.6		
0.0404 mm.	59.5		
0.0303 mm.	47.0		
0.0204 mm.	32.9		
0.0121 mm.	24.6		
0.0088 mm.	20.9		
0.0063 mm.	18.4		
0.0045 mm.	16.8		
0.0032 mm.	15.6		
0.0023 mm.	13.1		
0.0013 mm.	11.4		

* (no specification provided)

<u>Soil Description</u>		
Olive-brown SILT with SAND		
<u>Atterberg Limits</u>		
PL= 23	LL= 27	PI= 4
<u>Coefficients</u>		
D ₈₅ = 0.0996	D ₆₀ = 0.0410	D ₅₀ = 0.0325
D ₃₀ = 0.0179	D ₁₅ = 0.0029	D ₁₀ =
C _u =	C _c =	
<u>Classification</u>		
USCS= ML	AASHTO=	
<u>Remarks</u>		

Sample No.: 14
Location:

Source of Sample: B-3

Date: 6/4/04
Elev./Depth: 59'

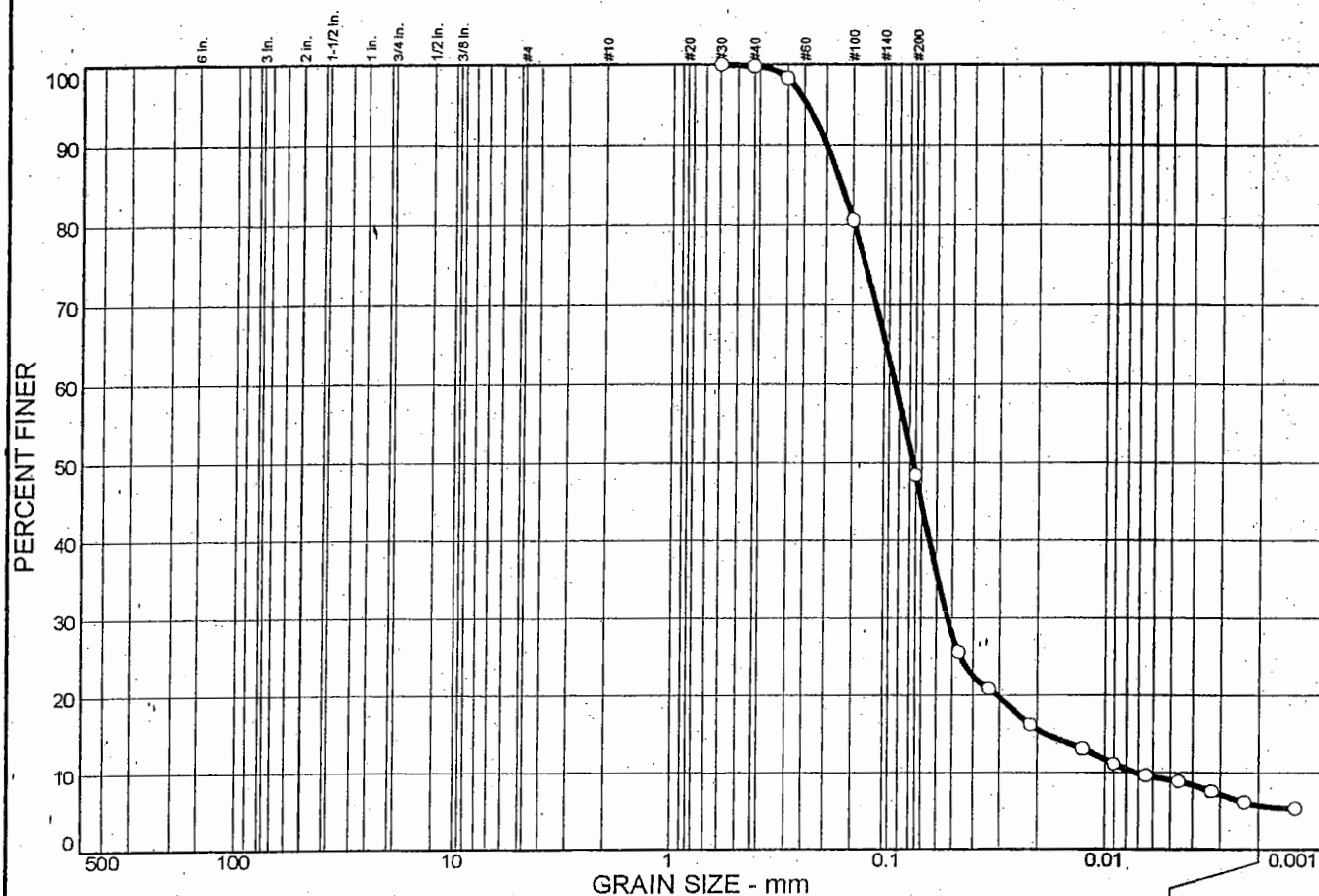
COOPER TESTING LABORATORY

Client: Treadwell & Rollo
Project: Milpitas Library - 3918.01

Project No: 010-831

Figure

PARTICLE SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.2	51.4	42.7	5.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#30	100.0		
#40	99.8		
#50	98.3		
#100	80.6		
#200	48.4		
0.0466 mm.	25.6		
0.0336 mm.	20.9		
0.0216 mm.	16.2		
0.0126 mm.	13.1		
0.0091 mm.	11.1		
0.0065 mm.	9.6		
0.0046 mm.	8.8		
0.0033 mm.	7.5		
0.0023 mm.	6.1		
0.0014 mm.	5.3		

* (no specification provided)

Soil Description
Olive Silty SAND

Atterberg Limits
PL= 24 LL= PI= NP

Coefficients
D₈₅= 0.170 D₆₀= 0.0936 D₅₀= 0.0772
D₃₀= 0.0527 D₁₅= 0.0184 D₁₀= 0.0073
C_u= 12.89 C_c= 4.09

Classification
USCS= SM AASHTO=

Remarks

Sample No.: 15

Location:

Source of Sample: B-3

Date: 6/4/04

Elev./Depth: 70'

COOPER TESTING LABORATORY

Client: Treadwell & Rollo
Project: Milpitas Library - 3918.01

Project No: 010-831

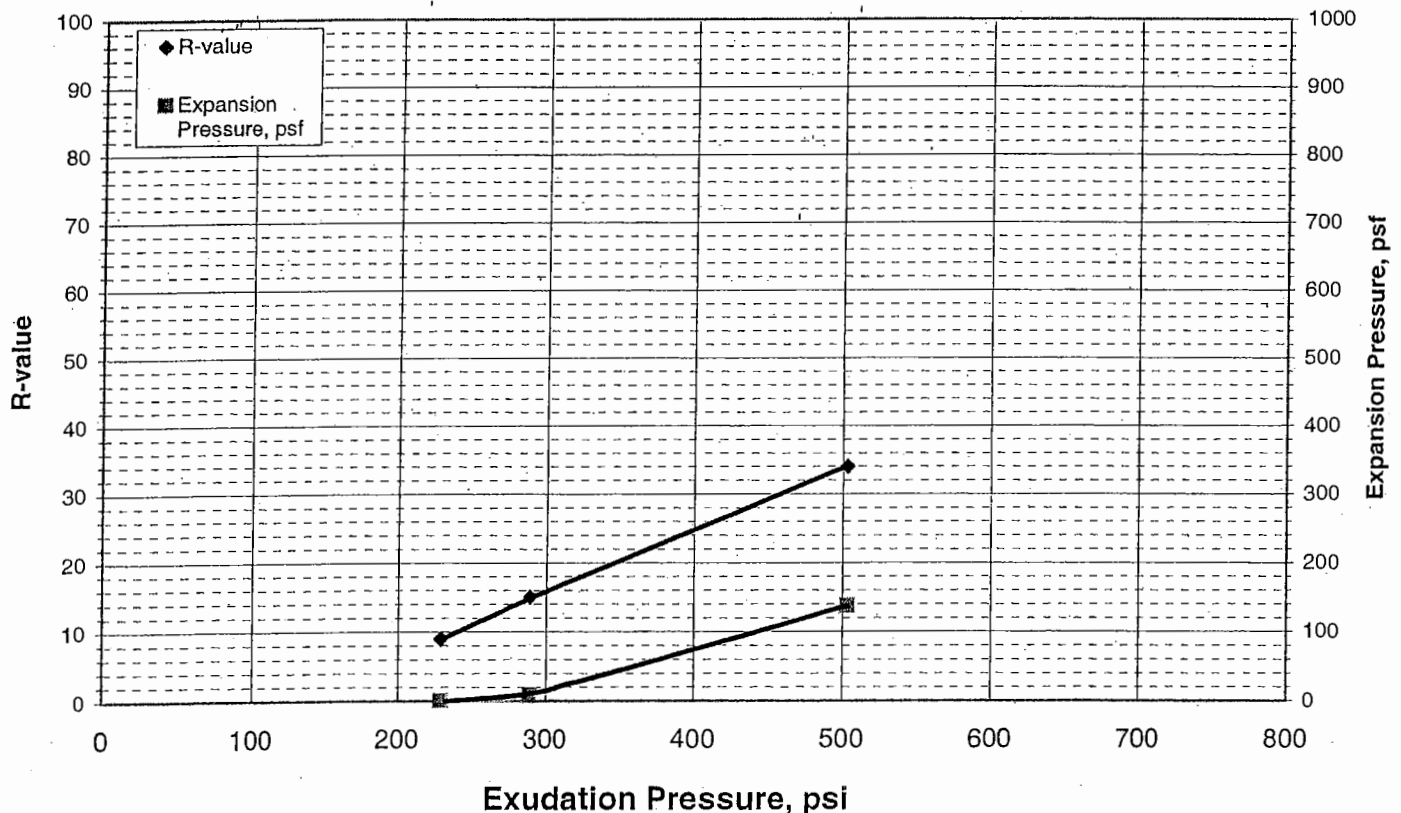
Figure



R-value Test Report (Caltrans 301)

Job No.: 010-831a	Date: 05/24/04	Initial Moisture, 10.7%
Client: Treadwell & Rollo	Tested MD	R-value 16
Project: Milpitas Library - 3918.01	Reduced MJ	Expansion Pressure 20 psf
Sample B-1, Bulk @ 0.5-3"	Checked DC	
Soil Type: Dark-gray Clayey SAND, trace Gravel		

Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	289	229	503		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	40	60	20		
Weight of Soil & Mold, grams	3226	3176	3178		
Weight of Mold, grams	2108	2096	2075		
Height After Compaction, in.	2.53	2.45	2.45		
Moisture Content, %	14.4	16.3	12.6		
Dry Density, pcf	116.9	114.8	121.1		
Expansion Pressure, psf	8.6	0.0	137.6		
Stabilometer @ 1000					
Stabilometer @ 2000	130	141	101		
Turns Displacement	3.38	3.6	2.87		
R-value	15	9	34		

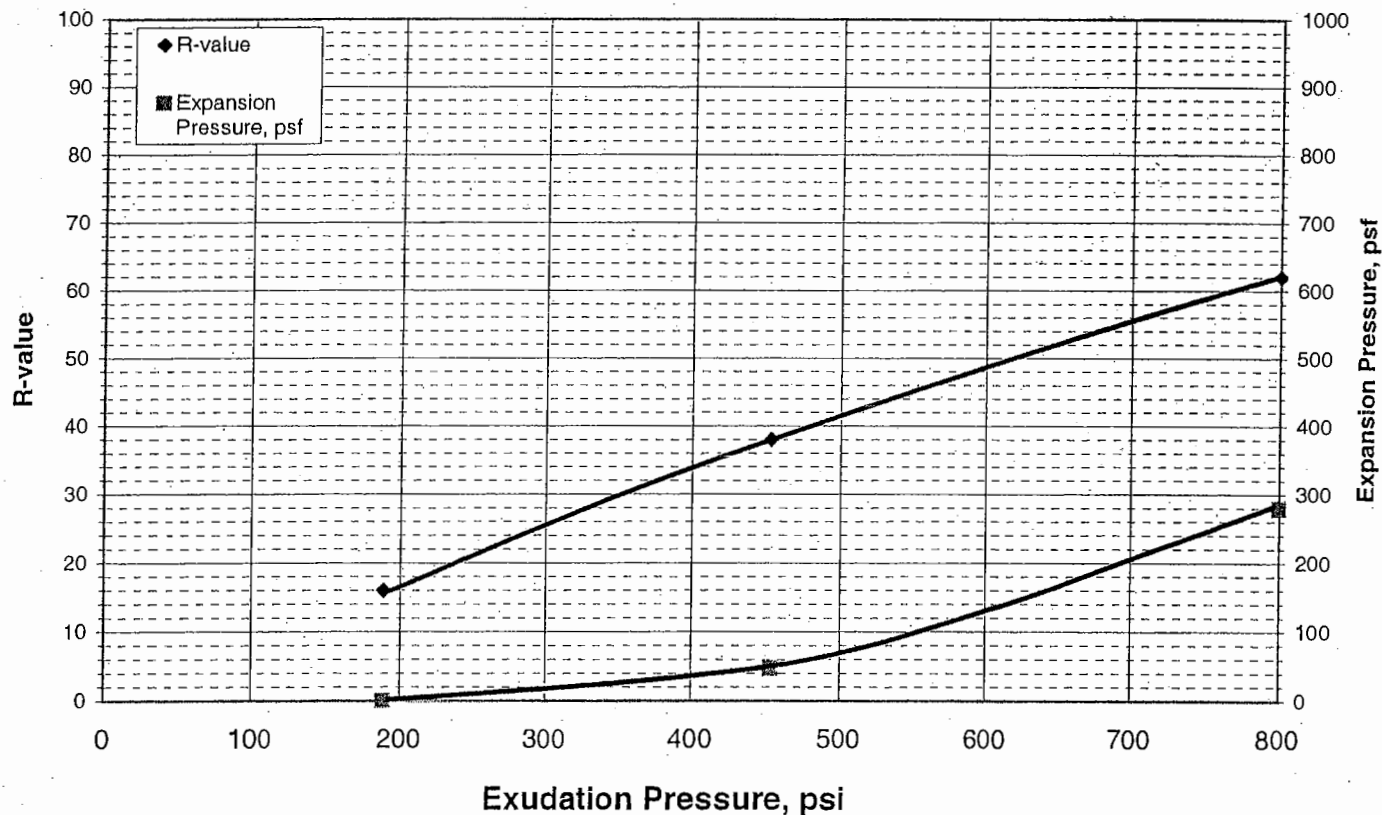




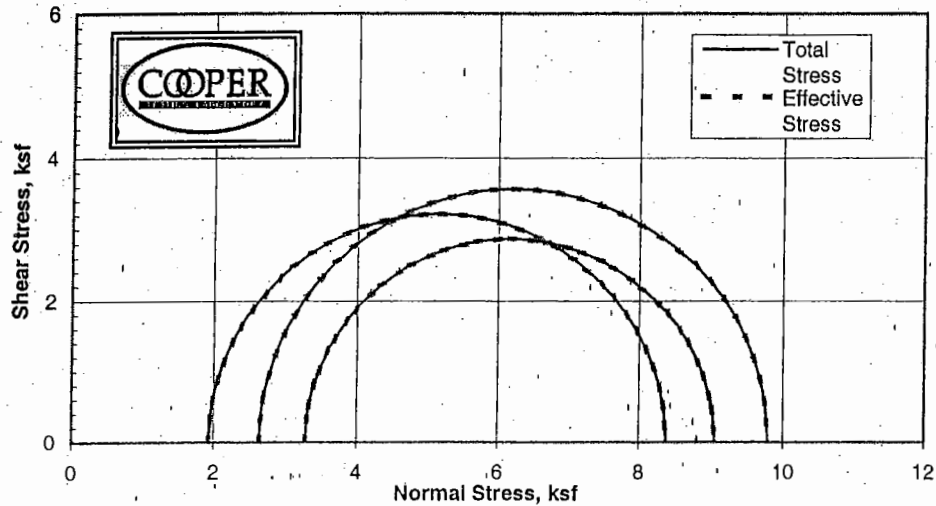
R-value Test Report (Caltrans 301)

Job No.: 010-831b	Date: 05/21/04	Initial Moisture, 10.3%
Client: Treadwell & Rollo	Tested MD	R-value 26
Project: Milpitas Library - 3918.01	Reduced MJ	Expansion Pressure 20 psf
Sample B-3, Bulk @ 0.5 - 2.5'	Checked DC	
Soil Type: Brown Clayey SAND, trace Gravel		

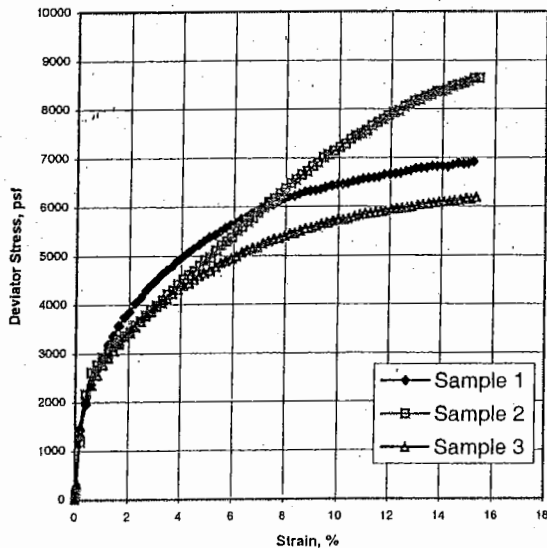
Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	189	453	800		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	40	20	0		
Weight of Soil & Mold, grams	3260	3210	3217		
Weight of Mold, grams	2095	2075	2070		
Height After Compaction, in.	2.58	2.47	2.45		
Moisture Content, %	14.0	12.2	10.3		
Dry Density, pcf	119.9	124.0	128.5		
Expansion Pressure, psf	0.0	47.3	279.5		
Stabilometer @ 1000					
Stabilometer @ 2000	128	91	54		
Turns Displacement	3.62	3.1	2.95		
R-value	16	38	62		



Triaxial Consolidated Undrained



Stress-Strain Response



Sample:	1	2	3	4
MC, %	19.0	24.8	21.9	
DD, pcf	112.9	103.6	109.5	
Sat. %	97.3	101.2	102.7	
Void Ratio	0.547	0.687	0.596	
Diameter in	2.41	2.41	2.41	
Height, in	5.00	5.00	5.00	
Final				
MC, %	19.9	22.8	22.2	
DD, pcf	112.1	106.6	107.7	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.558	0.639	0.623	
Diameter, in	2.42	2.39	2.44	
Height, in	4.99	4.93	4.94	
Cell, psi	62.3	66.6	72.1	
BP, psi	48.9	48.3	49.3	
Effective Stresses At:				
Strain, %	10.0	10.0	10.0	
Deviator ksf	6.447	7.142	5.749	
Excess PP	0.000	0.000	0.000	
Sigma 1	8.376	9.778	9.032	
Sigma 3	1.930	2.635	3.283	
P, ksf	5.153	6.206	6.158	
Q, ksf	3.223	3.571	2.875	
Stress Ratio	4.341	3.710	2.751	
Rate in/min	0.002	0.002	0.002	

Job No.: 010-831 Date: 38037

Client: Treadwell & Rollo BY:DC

Project: 3918.01

Sample 1) B1-13 @ 45' Olive CLAY

Sample 2) B6-14 @ 60' Olive-gray Silty CLAY

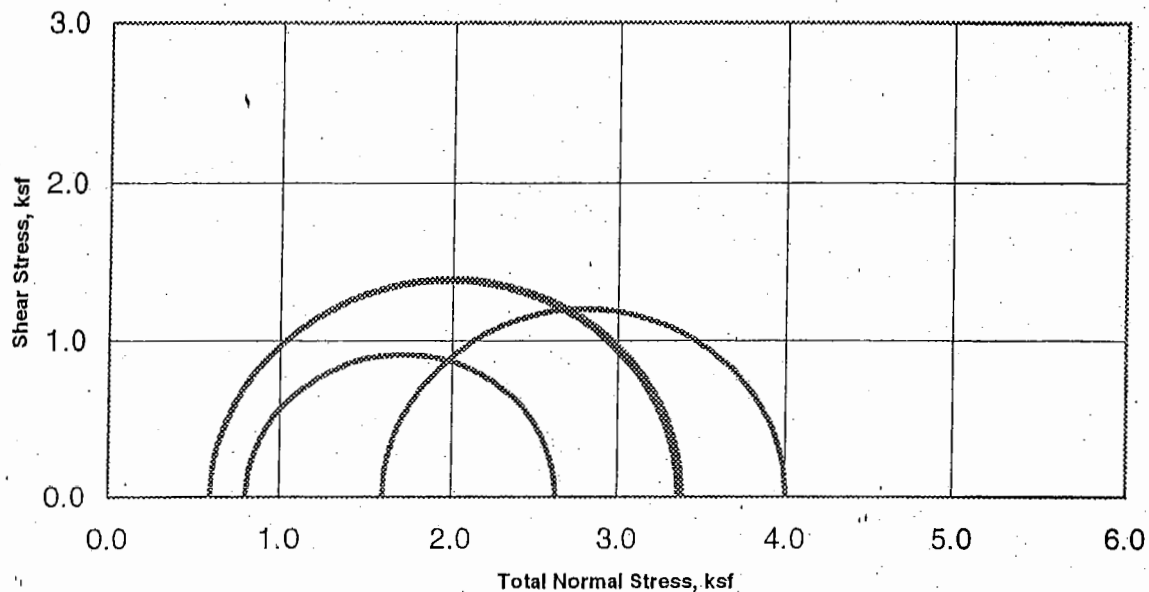
Sample 3) B5-14 @ 80' Olive-brown CLAY

Sample 4)

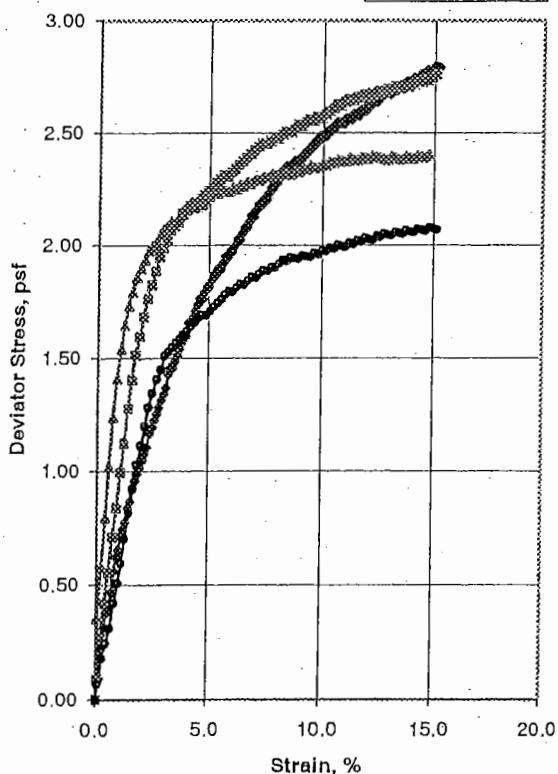
Remarks: Values picked at 10% strain.



Unconsolidated-Undrained Triaxial Test ASTM D-2850



Stress-Strain Curves



Sample Data

	1	2	3	4
Moisture %	18.7	20.9	24.5	28.6
Density pcf	113.1	107.3	101.0	95.9
Void Ratio	0.519	0.601	0.699	0.790
Saturation %	99.4	95.8	96.2	99.6
Height in	5.00	6.00	6.00	6.00
Diameter in	2.42	2.87	2.87	2.88
Cell psi	4.2	4.2	11.1	5.6
Strain %	14.90	15.10	12.40	14.80
Deviator, ksf	2.798	2.760	2.405	2.078
Rate %/min	1.00	1.00	1.00	1.00
In/min	0.050	0.060	0.060	0.060

Job No.:	010-831a			
Client:	Treadwell & Rollo			
Project:	Milpitas Library - 3918.01			
Boring:	B-1	B-2	B-3	B-4
Sample:	4	4	10	6
Depth ft:	10	9	34	14

Visual Soil Description

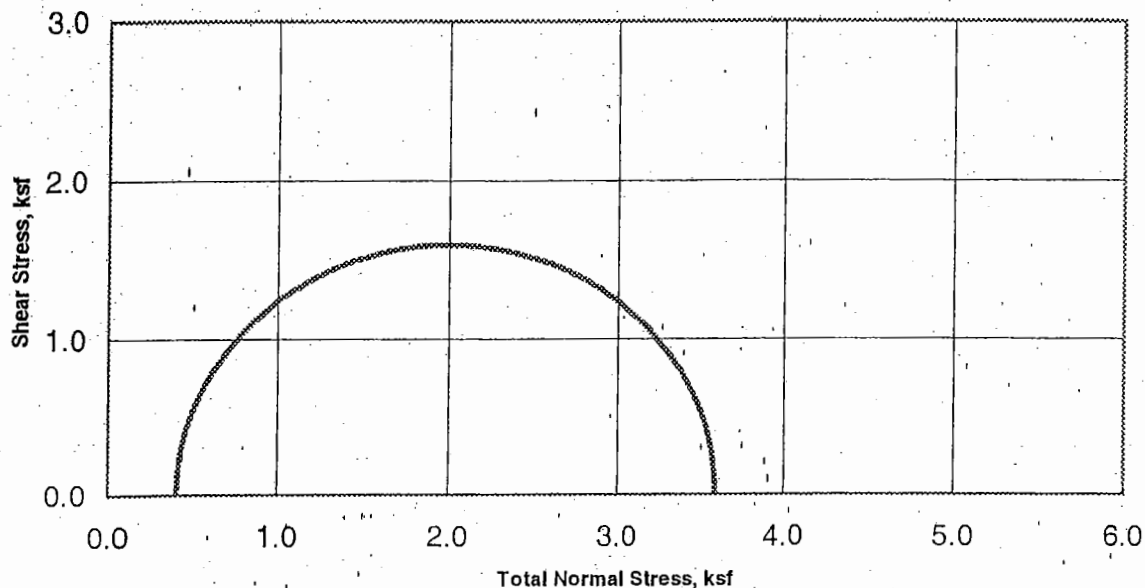
Sample #	
1	Olive CLAY with Sand
2	Olive-gray Sandy CLAY (silty), trace Gravel
3	Olive-brown Sandy CLAY (silty)
4	Olive-brown CLAY

Remarks:

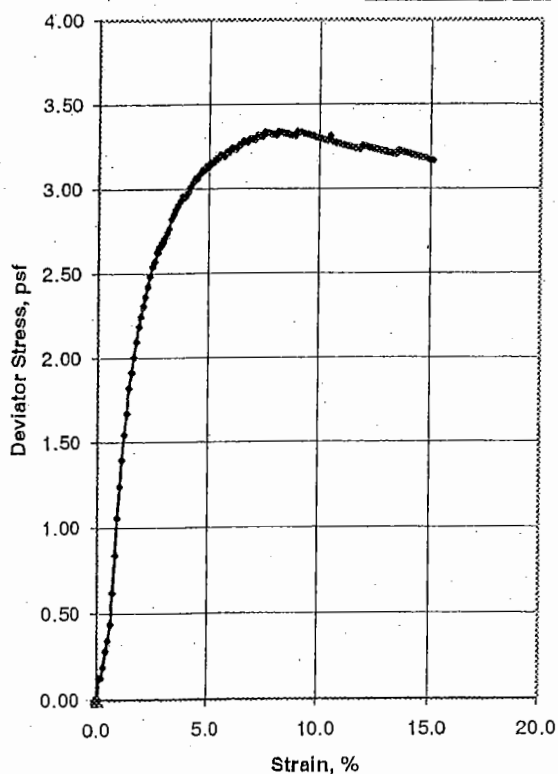


Unconsolidated-Undrained Triaxial Test

ASTM D-2850



Stress-Strain Curves



Sample Data

	1	2	3	4
Moisture %	29.9			
Density pcf	93.7			
Void Ratio	0.831			
Saturation %	98.8			
Height in	5.00			
Diameter in.	2.42			
Cell psi	2.8			
Strain %	8.10			
Deviator, ksf	3.339			
Rate %/min	1.00			
in/min	0.050			
Job No.:	010-831b			
Client:	Treadwell & Rollo			
Project:	Milpitas Library - 3918.01			
Boring:	B-6			
Sample:	2			
Depth ft:	5			

Visual Soil Description

Sample #	
1	Gray-brown CLAY
2	
3	
4	

Remarks:



Consolidation Test

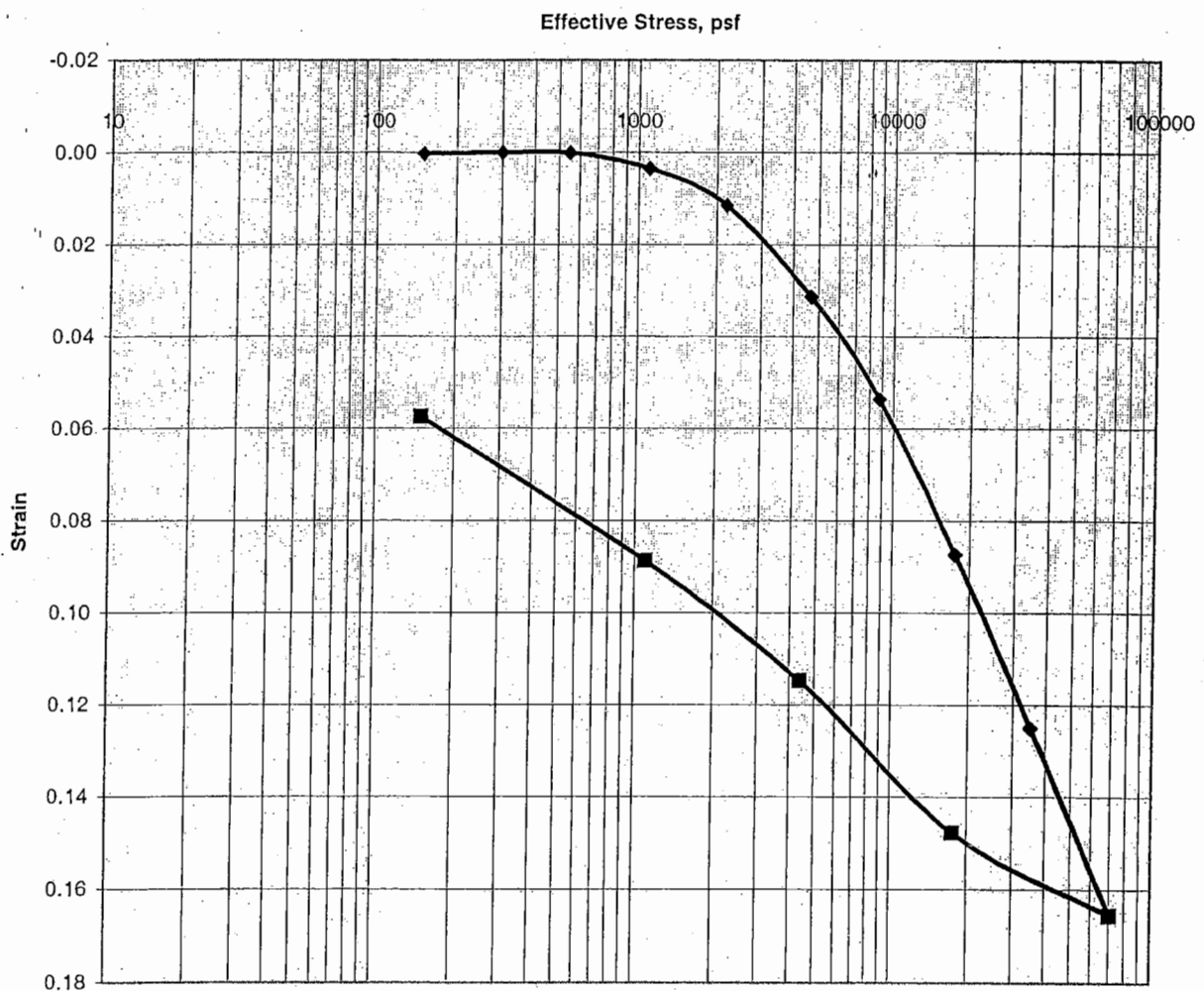
ASTM D2435

Job No.: 010-831a
 Client: Treadwell & Rollo
 Project: 3918.01
 Soil Type: Olive CLAY w/Sand

Boring: 1
 Sample: 3
 Depth': 6'

Run By: MD
 Reduced: DC
 Checked: DC
 Date: 6/7/2004

Strain-Log-P Curve



Ass. Gs = 2.7	Initial	Final
Moisture %:	25.1	24.4
Density, pcf:	99.7	101.7
Void Ratio:	0.691	0.657
% Saturation:	98.2	100

Remarks:



Consolidation Test

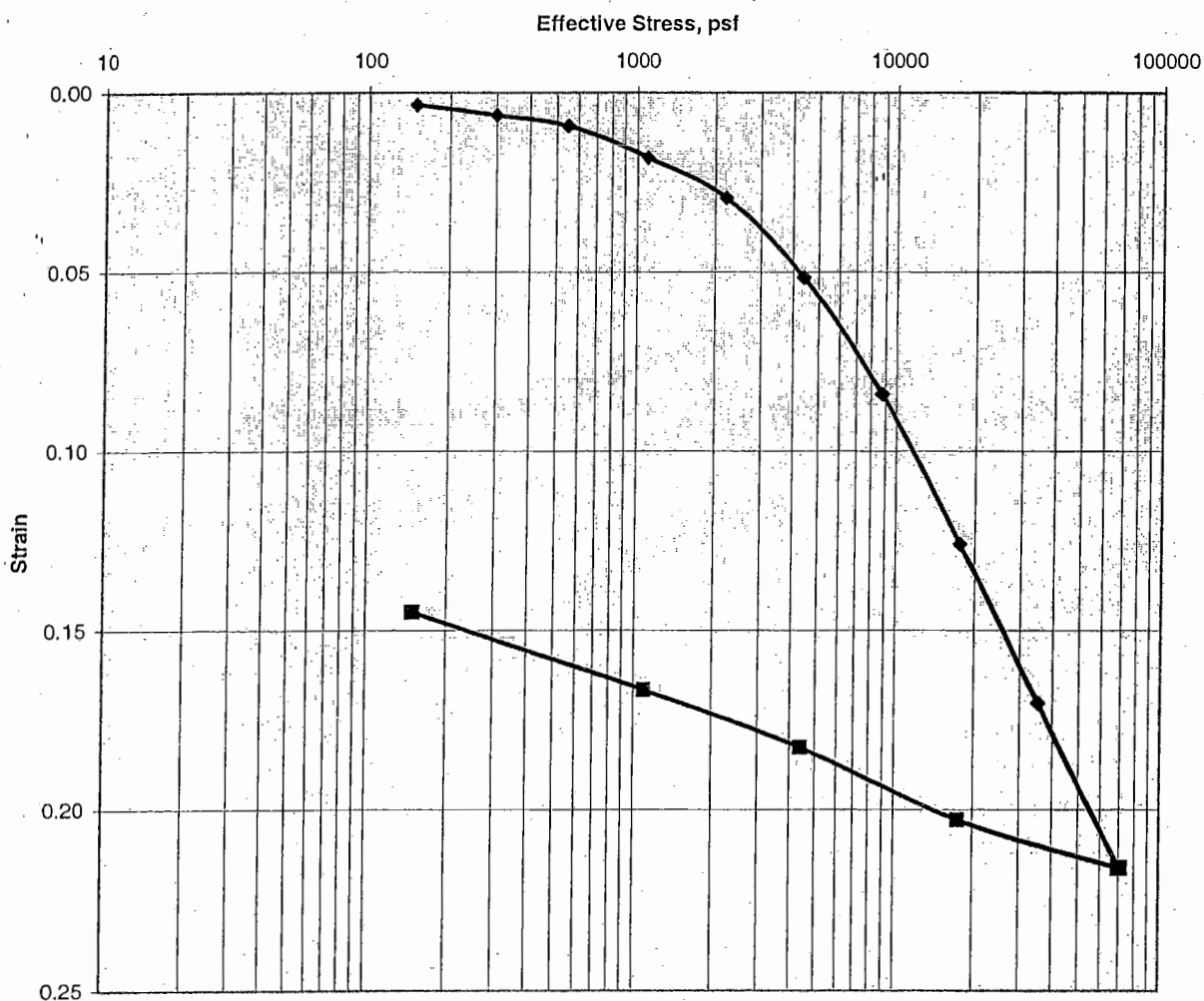
ASTM D2435

Job No.: 010-831
 Client: Treadwell & Rollo
 Project: 3918.01
 Soil Type: Olive-brown CLAY

Boring: 4
 Sample: 6
 Depth': 14'

Run By: MD
 Reduced: DC
 Checked: DC
 Date: 6/7/2004

Strain-Log-P Curve



Ass. Gs = 2.7	Initial	Final
Moisture %:	30.6	24.4
Density, pcf:	89.9	101.7
Void Ratio:	0.876	0.658
% Saturation:	94.3	100

Remarks:



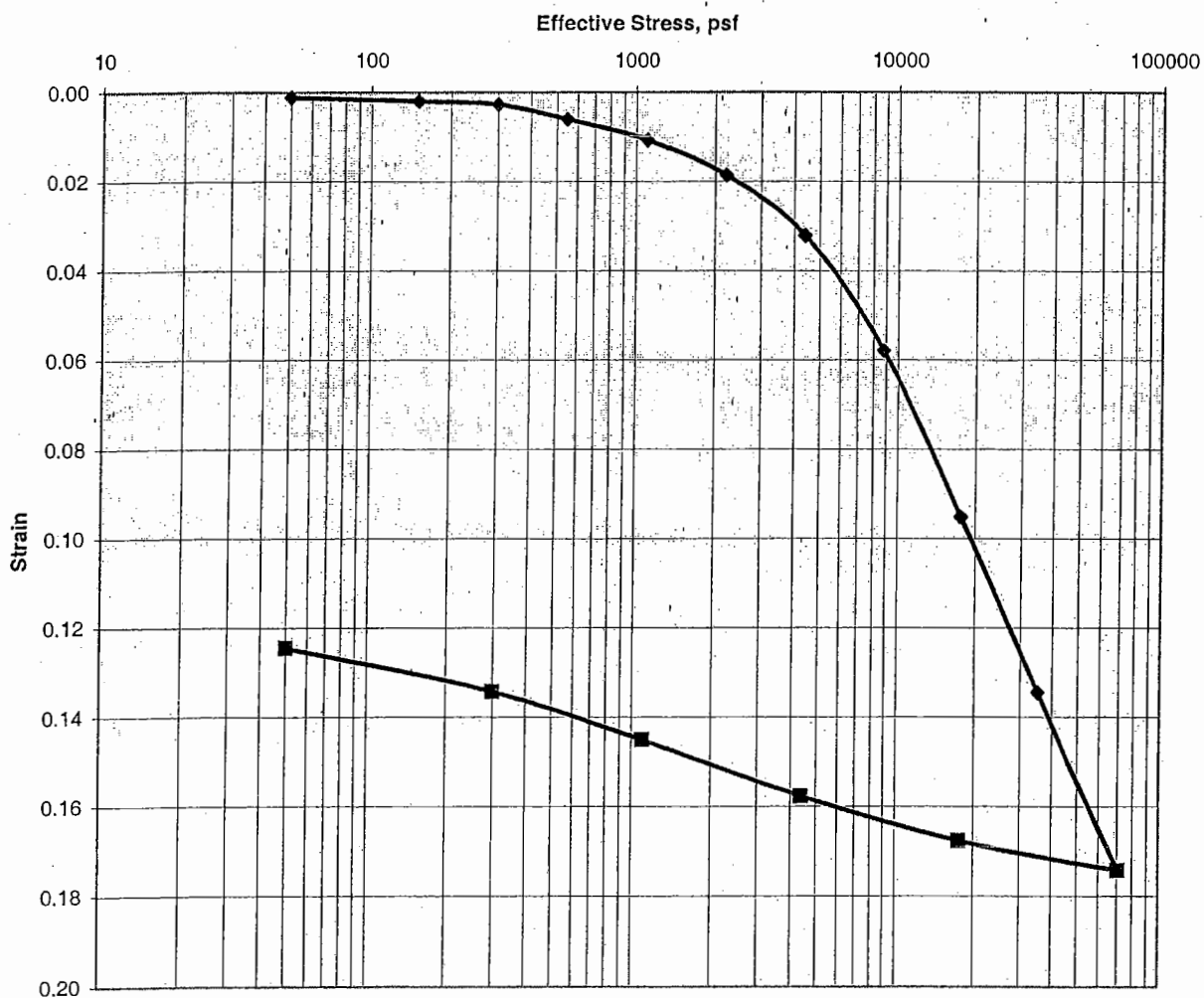
Consolidation Test

ASTM D2435

Job No.: 010-831
 Client: Treadwell & Rollo
 Project: 3918.01
 Soil Type: Olive-brown Sandy CLAY

Boring: 3
 Sample: 10
 Depth': 34'
 Run By: MD
 Reduced: DC
 Checked: DC
 Date: 6/7/2004

Strain-Log-P Curve



Ass. Gs =	2.7	Initial	Final
Moisture %:		24.5	19.0
Density, pcf:		98.7	111.4
Void Ratio:		0.707	0.513
% Saturation:		93.5	100

Remarks:



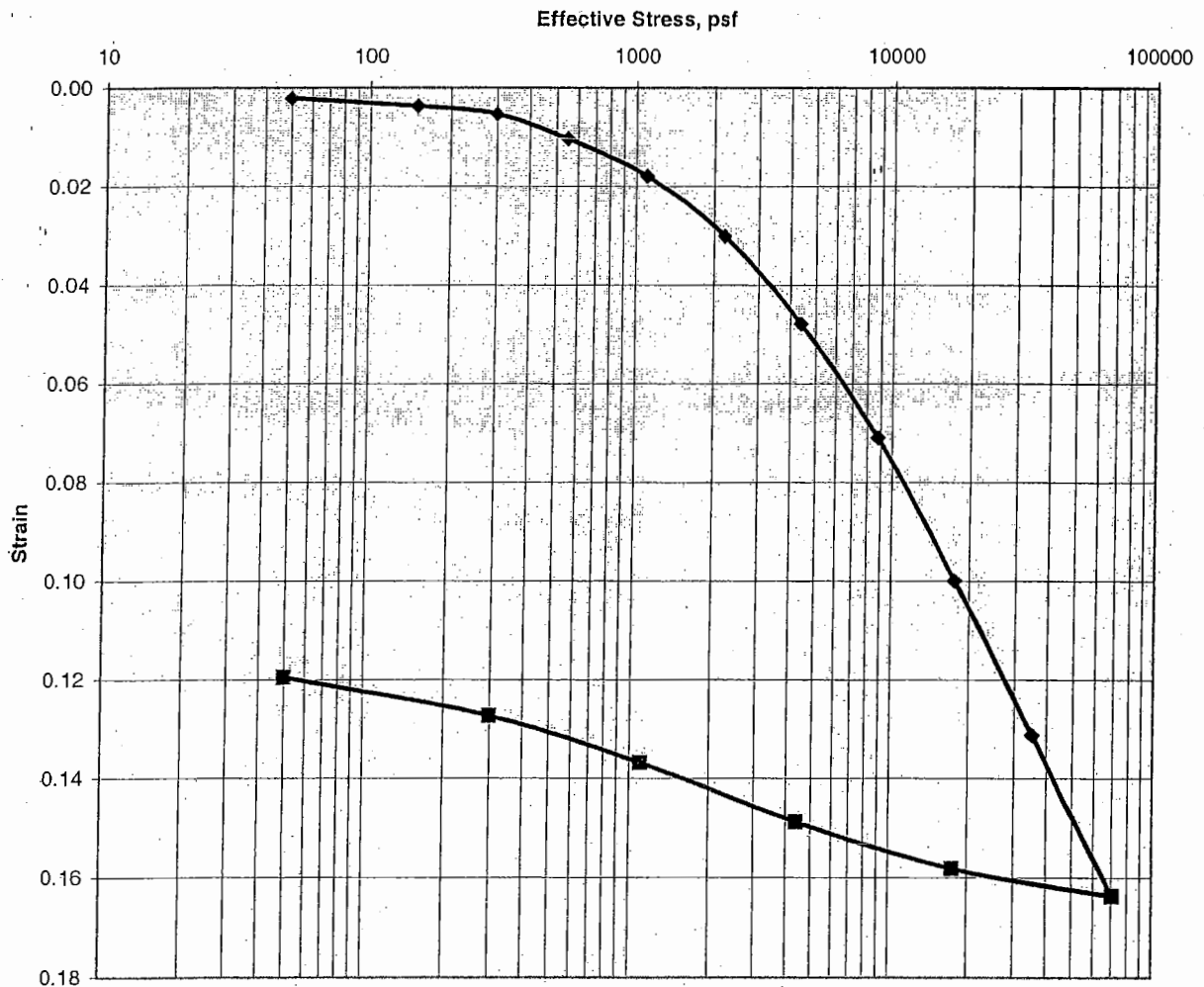
Consolidation Test

ASTM D2435

Job No.: 010-831
 Client: Treadwell & Rollo
 Project: 3918.01
 Soil Type: Olive-gray Sandy CLAY, (silty)

Boring: 2
 Sample: 4
 Depth': 9'
 Run By: MD
 Reduced: DC
 Checked: DC
 Date: 6/7/2004

Strain-Log-P Curve



Ass. Gs =	2.7	Initial	Final
Moisture %:		21.0	16.5
Density, pcf:		104.7	116.6
Void Ratio:		0.610	0.445
% Saturation:		92.8	100

Remarks: